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ARIZONA SECTION
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FOREWORD

During the first Seepage Symposium in 1963, the participants felt that later symposia should be held to review current progress and to discuss additional aspects of seepage. The first Seepage Symposium was arranged and conducted by the U. S. Water Conservation Laboratory. Widespread interest, greatly exceeding expectations, resulted in sponsorship of the second Seepage Symposium by the Irrigation and Drainage Division, American Society of Civil Engineers; the University of Arizona; the Arizona Section, American Society of Agricultural Engineers; and the U. S. Water Conservation Laboratory.

The second symposium was designed to provide a means of communication among research workers, engineers and technicians concerned with field problems, and representatives of industry. Excellent attendance and participation by representatives from all these groups indicated that the objective was reasonably achieved.

Any success the symposium may have attained was due to the efforts of many people, including the program committee, the session chairmen, and the authors of the various papers. The program committee members were Kenneth K. Barnes and Sol D. Resnick, University of Arizona; and Herman Bouwer, Robert J. Reginato, and Lloyd E. Myers, U. S. Water Conservation Laboratory. These men also represented the American Society of Civil Engineers and the American Society of Agricultural Engineers. Maurice N. Langley, Kenneth K. Barnes, Amalio Gomez, and Tyler N. Quackenbush deserve commendation for the manner in which they guided presentations and discussions during the sessions they chaired. Special thanks are due Aleta Morse for editing the galley and page proofs. Finally, we must express our appreciation to the authors of the papers for sharing their information and opinions with us.

Program Chairman and Editor

April 1969

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SESSION IV — SEEPAGE DETECTION AND EVALUATION

Chairman: Tyler N. Quackenbush

Italic numbers in parentheses refer to Literature Cited at the end of each paper. The data presented in the references are essentially used as they were supplied by the writer of each paper.

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SESSION I. — SEEPAGE AND WATER QUALITY

Chairman: Maurice N. Langley

RELATIONSHIP OF WATER QUALITY AND SOIL PROPERTIES TO SEEPAGE PROBLEMS[1]

J. Harlan Glenn[2]

Soil linings will continue to be a part of water works construction and operation for many years, even though the so-called permanent linings are used more and more frequently. If applicable, soil linings are generally the most economical method of controlling seepage, especially in fairly large reservoirs and sewage lagoons. The problem, of course, is to know how efficient a soil lining will be before construction begins. After the rate of seepage loss allowable is determined, the physical properties of the available soils must be determined. The most difficult task, and the one often ignored, is to correlate the physical properties determined in the laboratory to the field conditions. Can the desired properties be achieved by normal construction methods? What effect will operating conditions have on the soil lining? And finally, what effect will the quality of the water conveyed or contained by the facility have on the soil lining during the life of the facility?

Reports from many field projects indicate that actual seepage loss rates in the field are often 5 to 10 times that predicted from standard laboratory soil tests. The reasons for these differences can be attributed generally to inability of the contractor to obtain the specified compaction, to operational procedures, or to the effect of water quality on the soil lining. The primary objective of this paper is to discuss what is known about the effect of water quality on seepage rates through soil linings, and to indicate what can be done about changing these rates.

The irrigated agricultural industry has long recognized the importance of the relationship between water quality and soil properties. In fact, the industry survival depends not only on the recognition of the problem, but also on the corrective measures taken. The primary purpose of this branch of soil science is to keep irrigated land open, free draining, and free of salt accumulation. At the 1963 Seepage Symposium, Bower (*1*) presented what is probably the most concise and clear summary of all the work that has been accomplished. In essence, the permeability of soils containing clay fractions is relatively low when in equilibrium with a water containing a high ratio of sodium to calcium plus magnesium, but relatively high when in equilibrium with a water containing a low ratio of sodium to calcium plus magnesium, and an increase in total salt concentration generally tends to increase permeability. Detailed procedures for classifying irrigation waters use these principles extensively so that the water user knows what corrective treatments are necessary to prevent damage.

The petroleum industry has extensively studied the chemistry of clays and water with relation to the physical properties of the clays. In drilling a well to produce oil, at least three factors involving clay and water must be evaluated.

First, clay slurries are widely used as a circulating fluid to support the walls of the hole, to carry out the cuttings, and to provide a hydrostatic head to overcome any high formation pressures encountered. Furthermore, the clay slurry must build a wall cake of low permeability on porous or permeable zones.

Almost all of the slurries are prepared with Wyoming bentonite, having approximately

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[1] Contribution from Seepage Control Inc., Phoenix, Ariz.

[2] Vice president and chief engineer.

a 70 percent sodium saturation. Relatively small amounts of salts from makeup water or from the formation drilled can drastically affect the rheological and wall-building properties of the clay slurry. In general, small amounts of sodium salts, particularly sodium phosphates and tannates, tend to disperse the muds or lower the viscosity and to reduce the permeability of the clay-mud cakes, whereas small amounts of calcium salts, such as gypsum or calcium chloride, tend to increase the permeability of the cakes and adversely increase the apparent viscosity of the slurry.

Second, a porous producing horizon that contains clay could become plugged with swelled clay particles if the mud filtrate has a high sodium to calcium plus magnesium ratio.

Third, shale formations that are predominately clay can swell and shrink when wet with muds, thereby causing hole stability problems. Recognition of these factors influences the design of the mud and casing program in order that holes can be drilled safely and economically while protecting the producing horizon from damage.

Unfortunately, the knowledge gained in these two fields of science is not widely used in water-works design. For example, soils from a prospective canal or reservoir are sent to a laboratory for analysis, and tests are made for particle size distribution, optimum moisture and density, and water permeability. The particle size of the minus 200 mesh soil or fines is generally determined in a sodium phosphate solution so that the clay fraction is deflocculated and dispersed. The water used for the moisture-density relationship and the permeability tests is generally distilled or laboratory tap water, but it is seldom noted in the report. The permeability is generally reported as water permeability in centimeters per second or feet per year under unit gradient at optimum density, without reference to the water used. The results of these tests are used by the civil engineer to specify the thickness and density of the lining for a particular soil.

At about this point, the soils are usually classed as pervious or impervious, and the permeability coefficient so laboriously obtained is forgotten. What do we mean by pervious and impervious? To me, impervious would mean no seepage loss. Apparently, many others do not accept this definition. For example, at the 1963 Seepage Symposium, a seepage loss of 0.5 c.f.d. was reported for impervious clay in a southern Idaho canal (*3*). If reports such as this were rare, they could be dismissed as inadvertent errors in soil classification, but unfortunately, such reports are common. Probably, under the conditions of tests, the soil was impervious for the practical purpose of the person or agency making the test, but under field conditions, the soil was not impervious, and should not be classed as such. What happens between the laboratory tests and actual field conditions, and what can be done about it? Some examples will help illustrate the variety of factors.

The soil on a recently proposed artificial lake in Phoenix contained approximately 13 percent clay, 50 percent silt, and 37 percent sand, which would be a silt loam under the USDA soil classification system. The lake was to be filled with Phoenix tap water to a maximum depth of 4 feet. The optimum Proctor density was 113 PCF (pounds per cubic foot), and a field density of 108 PCF of 95 percent plus was easily achieved. The soil, however, expanded when wet with Phoenix tap water, and the field density after wetting and drying was 91 PCF. A series of laboratory permeability tests at 91 PCF dry density using Phoenix tap water and modifications thereof indicated a permeability coefficient range of 0.01 to 1.08 feet per day under unit gradient as seen in figure 1. The tap water normally contains about 700 to 800 p.p.m. total dissolved solids and has a sodium to calcium plus magnesium ratio of about 0.4. Use of this water resulted in a permeability coefficient of 0.7 foot per day. Sodium phosphate treatment of this water reduced the coefficient to 0.01 foot per day. The addition of 0.5 percent calcium chloride to this water decreased the sodium to calcium ratio and increased the permeability coefficient to 1.08 feet per day. Lime softening of the water reduced the permeability coefficient to 0.22 foot per day. These results can be predicted to some extent from particle size tests conducted in the actual type of water to be used in the facility, such as shown in figure 2. In essence, this is the reverse of the dispersion ratio mentioned by Decker (*2*) at the 1963 symposium. If sufficient loading is placed on the soil to prevent the initial bulk swelling, the permeability coefficient is materially decreased. The permeability of this soil at various dry densities with Phoenix tap water under sufficient loading to prevent bulk swelling is shown in figure 3.

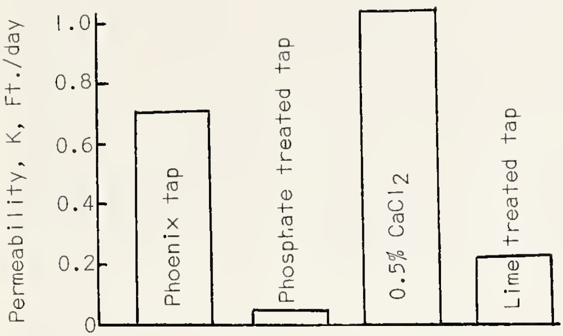

Figure 1.

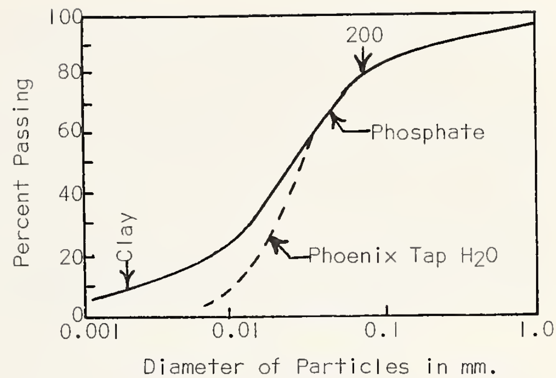

Figure 2.

This is not an isolated case, but typical of widely scattered areas, especially where montmorillonite clays are found. Another example would be the U. S. Department of the Interior, Bureau of Sport Fisheries and Wildlife fish hatchery ponds at Gavins Point, S. Dak. The soil involved was a silty clay as shown in figure 4. The Missouri River water,

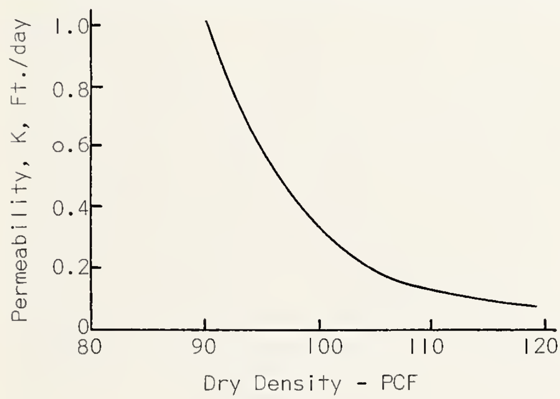

Figure 3.

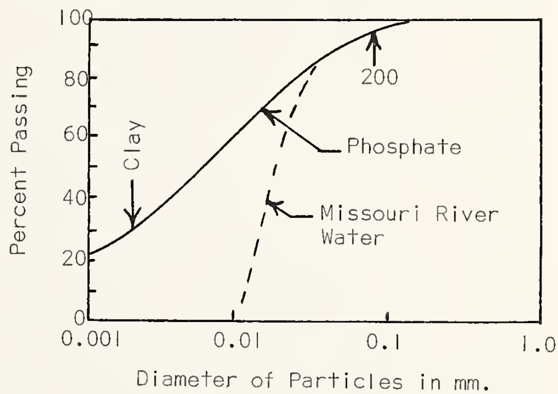

Figure 4.

like the Phoenix tap water, has a low sodium to calcium plus magnesium ratio, although the total dissolved solids are lower. The divergence between the phosphate-treated and field water soil curves in the standard soil classification test is greater because of the higher clay content of this soil compared with the soil previously mentioned. Twenty ponds, each 1.5 acres in size, were built using an 18-inch compacted layer of this soil, based on conventional short-term laboratory tests indicating that this soil had a very low permeability and would show little or no seepage. About a year after construction, these ponds were losing about one foot of water per day through seepage. In-place density tests showed densities less than those specified in the construction plans, so the soil in the ponds was reworked and recompacted. The seepage rate, however, was again in the 1 foot per day range within a short time. At this point, I was asked to investigate the seepage problem and make recommendations. Three factors combined to produce this seepage. The swelling of the soil was followed by aggregation of the clay particles by the calcium and magnesium salts in the water. In addition, the ponds were allowed to dry out between crops of fish, resulting in both vertical cracking and further aggregation of the clay lining.

The problem, of course, was to find a way of correcting all the factors involved. The chemical aggregation of the soil could be corrected by adding sodium salts to the soil, as reported by Decker (2) at the 1963 Seepage Symposium, or by altering the flocculating tendencies of the water by adding a dispersing agent such as a sodium phosphate as shown in figure 5, but such treatments would tend to increase the swelling properties and the cracking in a dry period. Apparently, the best solution would be one in which the swell-

ing tendency of the soil was eliminated. Since this swelling tendency results from the formation of water layers with nonliquid properties around the clay particles, the solution would be to eliminate or reduce the ability of the clay particles to adsorb water in an oriented manner. All clays, of course, do not exhibit the same capacity for oriented water adsorption, but this factor is evident even in silts. It is most evident in the montmorillonite or bentonitic clays, and extensive work has shown that the nature and extent of the oriented water development is a complex function of the cations adsorbed on the surface of the clay minerals. Adsorbed sodium ions favor the development of very thick oriented water layers, perhaps tens of molecular layers when sufficient water is available. There is no sharp break between oriented and nonoriented layers. But when the clays are air dried, sodium ions favor the development of a single molecular layer. Thus, sodium-saturated clays have very high swelling and shrinkage potentials.

On the other hand, calcium ions adsorbed on the surface of the clays tend to limit the development of oriented water to about four molecular layers, and there is a sharp break between the liquid and nonliquid water. Furthermore, in the air-dried state, calcium ions tend to favor oriented water layers two molecules thick. Thus, calcium-saturated clays have reasonably low swelling and shrinkage potentials. Magnesium ions have roughly the same effect as calcium ions.

Potassium, hydrogen, aluminum, and iron ions tend to produce similar features, but these effects are not as well known.

The gradual replacement of one cation by another may exhibit no influence on the adsorbed water arrangement until a critical point is reached where a sudden abrupt change in the thickness of the oriented water occurs, and this may be a stepwise change with resulting changes in volume and bonding strength between particles.

The clays in untreated soils may contain a dominant single adsorbed cation or a mixture of cations. A uniform oriented water layer may be prevalent if a single cation is present, and considerable stability will be present in the system which will resist a certain amount of cation exchange, especially when the soil is compacted to near optimum density. Most soils, however, have a variety of adsorbed cations and it is difficult to always predict when an abrupt change in properties could occur. In Wyoming bentonites, for example, an adsorbed cation composition of about 70 percent sodium and 30 percent calcium produces the maximum potential dispersibility or maximum swell and shrinkage. Salts in water contained or conveyed in a clay-lined structure offer a potential cation exchange source that can, over a period of time, alter the characteristics of the clay structure so that the design features are no longer present.

Therefore, the environment is extremely important in the design of a soil structure.

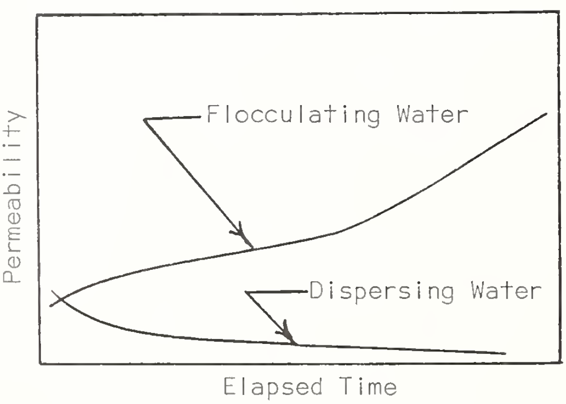

Figure 5.

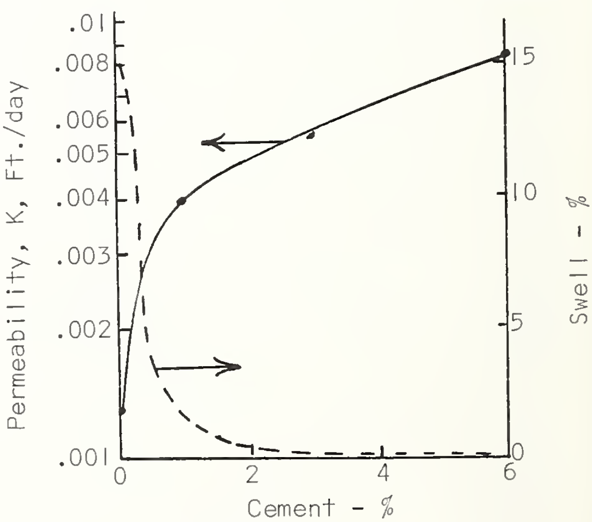

Figure 6.

In the fish-rearing ponds at Gavins Point, lime stabilization of the soil appeared to be the best solution to the problem of swelling, chemical aggregation, and wet-dry cycling, with sodium-phosphate treatment of the soil as second choice. I predicted in my report to the Bureau of Sport Fisheries that phosphate treatment of the soil would give a lower initial seepage loss than the lime treatment, but that the phosphate treatment efficiency would decrease rapidly while the lime treatment efficiency would increase with time. Field treatments were made using both methods, and the results were as predicted. An initial loss rate of 0.16 foot per day for the phosphate-treated soil under an 8-foot depth of water increased to 0.42 foot per day after 4 months' operation, including one dry cycle. For the lime-treated soil, the initial loss rate of 0.25 foot per day decreased to 0.10 foot per day after 4 months' operation, including a dry cycle. The lime treatment, even though higher in cost, appeared to give the lowest total control cost over a period of time. However, lime treatment is not practical for all soils and cannot be used as a universal treatment.

Swell potential can also be reduced with cement treatment. A good example of this was the design of a compacted liner for a brine evaporation pond. A clay containing about 25 percent gypsum was available for use, and since the brine water was already saturated with gypsum, the normally soluable gypsum content of the clay could be ignored, and the sodium chloride-saturated brine also provided a very high sodium ion reserve. However, the high swell potential changed the compacted layer into a soggy mess incapable of carrying salt-harvest equipment. Treatment with cement at rates greater than 2 percent by weight reduced the swell to essentially zero and provided a stable layer, and although the permeability was increased, it was still within acceptable limits. The relationship is shown in figure 6.

If desirable, the strength of the soil cement could be increased by the addition of chemicals based on clay studies. Such an increase might be desirable for reducing frost heave. Recent work at the University of Rhode Island shows what can be done with various salts, as seen in figures 7 and 8 for a silty soil *(7)*.

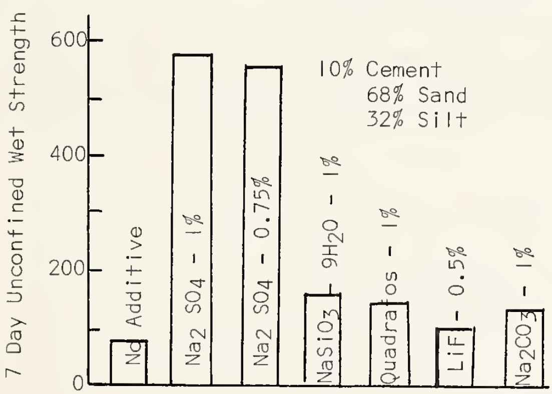

Figure 7.

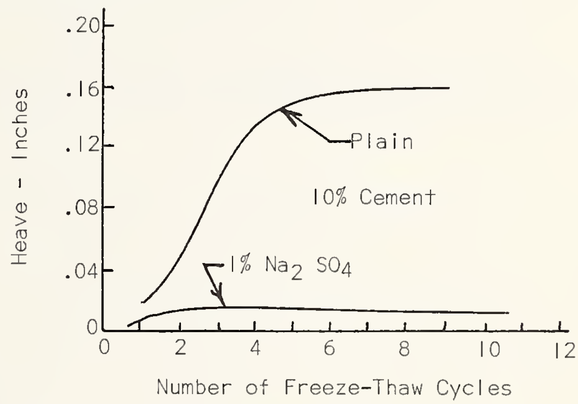

Figure 8.

Stabilization of a soil often will result in a permeability outside the desirable range for a given application. Studies of clay chemistry have shown that some adsorbed components, such as ionic organic materials, serve as a waterproof coating which not only helps prevent the development of water layers in a nonliquid state but actually waterproofs the soil. Personal work on this aspect since 1950 has been substantiated by independent work in Russia reported at the 1958 International Symposium on Water and Its Condition in Soils *(4)*. Considerable work has been done at Massachusetts Institute of Technology *(5, 6)*. Very small amounts of these ionic organic materials can change the soil properties considerably. For example, the change in consolidation under load of a soil from Taft, Calif., is shown in figure 9. The soil so treated is almost impermeable and has greatly reduced capillary rise of water. Good mixing of these materials with the soils is required for good results, and simple spraying of materials such as silicones on the surface of soils will not protect a fairly large mass of soil for any period of time. Also, not all soils are amenable to treatment.

Addition of certain materials as compaction aids can change the working properties of the soils so that certain densities are more easily achieved. For example, addition of dispersants such as sodium phosphates or aggregants such as sodium polyacrylates can change the moisture-density relationship of a given soil as shown in figures 10 and 11.

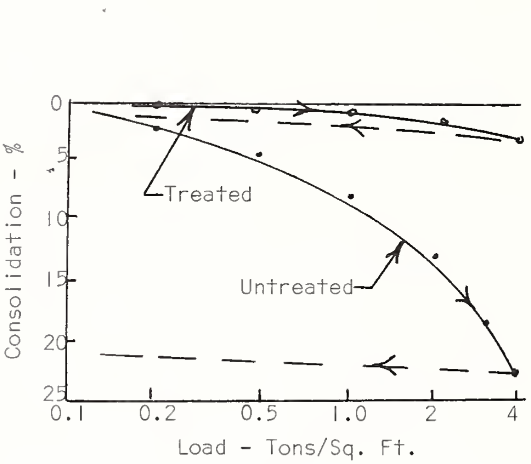

Figure 9.

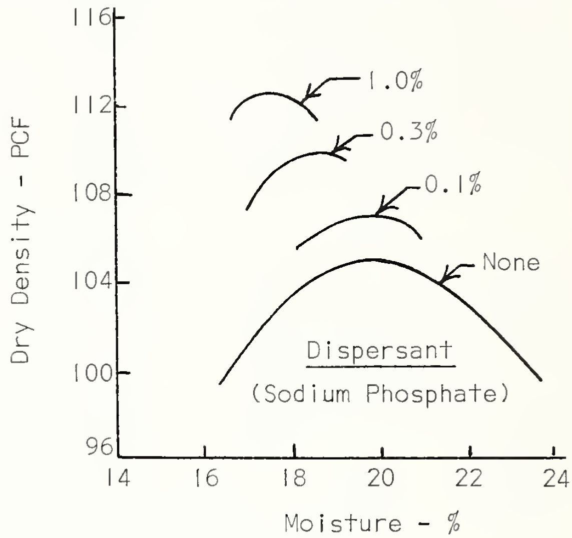

Figure 10.

However, only an understanding of the environment and soil chemistry can predict what such an addition will do to the ultimately desired soil properties.

Recognition of the changes in soil structure by environmental forces can also prevent costly mistakes in water containment and conveyance structures. Contrary to general opinion, most large structures such as long canals lined with our conventional construc-

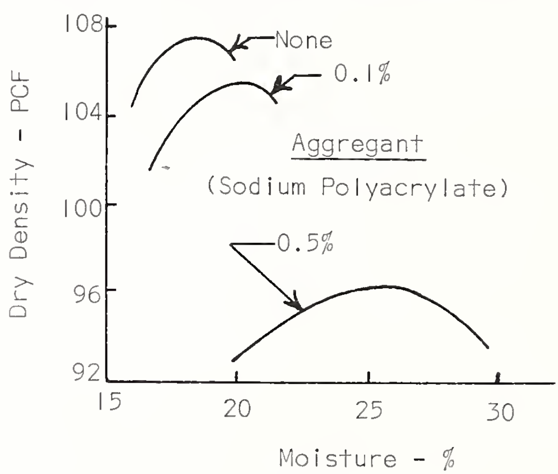

Figure 11.

tion materials do leak. A small amount of water reaching an expansive clay soil behind a concrete lining can destroy the lining unless compensating measures are made during construction.

In conclusion, environmental chemistry can enhance or destroy construction efforts in the water works field. This broad and relatively untouched field needs a lot of work. I hope that this brief and rather general discussion will help in creating interest in this task.

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CHANGES IN WATER QUALITY DURING SEEPAGE[1]

D. R. Nielsen and R. D. Jackson[2]

Quality usually implies a positive attribute or virtue. The term water quality, however, may connote either a good or bad aspect, depending upon the conditions under which the water is to be used. Water-quality criteria for domestic, agricultural, industrial, wildlife, and recreational uses differ considerably. Moreover, water-quality standards differ depending upon specific requirements and the reactions of water with other environmental constituents. On the other hand, the quality of water for all uses generally encompasses a quantitative measure of inorganic and organic solutes, dissolved gases, suspended materials — both liquid and solid, and biota. Less definitive considerations include taste, odor, and color. Recently, the goals of water-quality control agencies have expanded profoundly. No longer are discrete, individual uses emphasized, but research and management reflect water resource development and multipurpose objectives within our environment.

Regardless of the ecosystem considered, all water must eventually pass through the soil. Changes in water quality during this seepage phenomenon stem from complex physical-chemical and biochemical interactions. Chemical processes of oxidation, reduction, dissolution, and precipitation are well understood, particularly under equilibrium conditions. The interactions of solutes, water, and soil-particle surfaces involving absorption, adsorption, ionic exchange, and the chemical processes during transient soil-

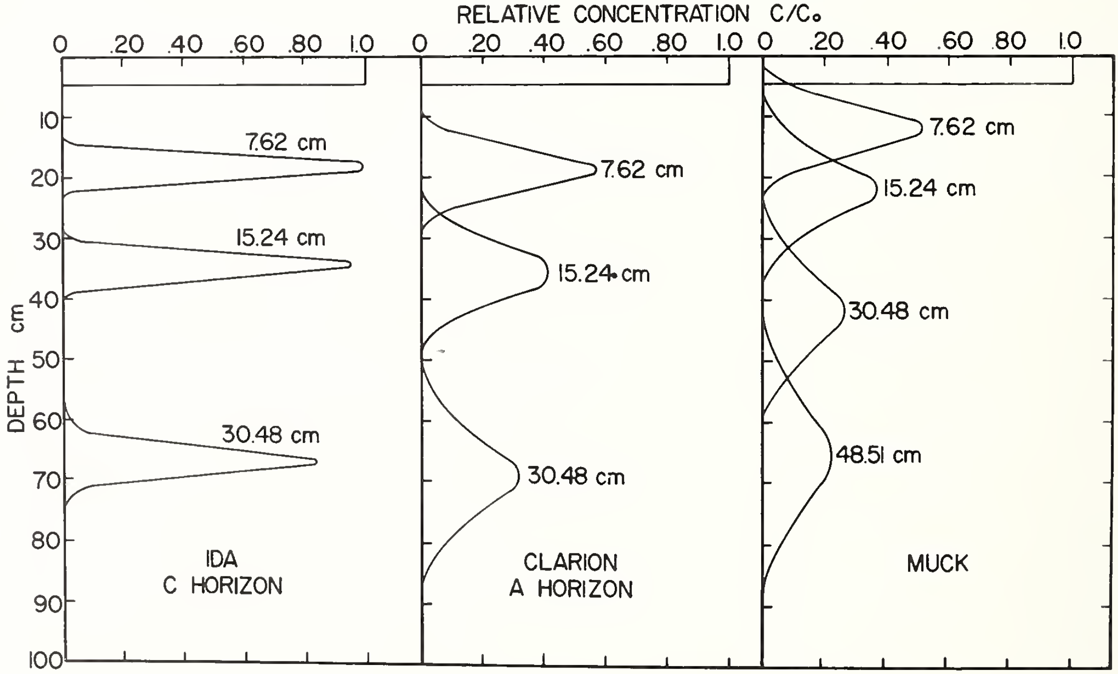

Figure 1.—The movement and relocation of a narrow band of solute initially at the soil surface during seepage. Numbers with each curve indicate the amount (cm.) of water which has entered the soil surface. Source: Corey, J. C., Kirkham, D., and Nielsen, D. R. The movement of chloride and nitrate through certain Iowa soils. Proceedings of Iowa Academy of Science. (In press.)

[1] Contribution from the Department of Water Science and Engineering, University of California, Davis, and the Soil and Water Conservation Research Division, Agricultural Research Service, U. S. Department of Agriculture.

[2] Associate professor of water science, University of California, Davis, and research physicist, U. S. Water Conservation Laboratory, Phoenix, Ariz., respectively.

water conditions are more difficult to understand and describe quantitatively. Biochemical processes involving specific biota and their microenvironment are just beginning to be studied in relation to water quality distributions within the soil profile. This paper is a terse review of the dynamic features of water quality within soils during seepage.

A major effort has been spent on the behavior of inorganic solutes in soil water during seepage, particularly the water-soluble salts related to soil salinity, soil fertility, and seawater intrusion into coastal aquifers. Technical journals contain many reports describing the dispersion or spreading of soluble salts within relatively simple, inert porous materials that remain water saturated during a constant seepage rate. These descriptions, summarized qualitatively in figure 1, illustrate some of the physical parameters important to changes in water quality during seepage.

Assume that a narrow band of soil solution containing a solute of C_0 concentration is leached through these three soil profiles by fresh water. Comparing the solute distribution curves for the Ida and Clarion soils (each having approximately equal water-filled porosities), 15.24 cm. of fresh water displaces the solute approximately to the same depth (35 cm.). But for the muck soil, having a much greater water-filled porosity, 15.24 cm. of water displaces the solute to a shallower depth (22 cm.). In all soils, the deeper the solute travels, the greater its spread throughout the profile. The tendency for the solute to spread or be dispersed differs among soils. In the Ida, the solute band, having been displaced to the 70-cm. depth, occupies only 10 cm. of the profile. For Clarion, the solute band at this depth is spread over 30 cm., and for the muck, over 40 cm. At this 70-cm. depth, the maximum concentration in the Ida is 0.8 and that in the muck about 0.2. This spreading action is the result of water moving at different rates through pores of different sizes. The rate of solute movement by molecular diffusion also aids the spreading process. The solute behavior depicted in figure 1 does not include chemical, physical, or biological processes that would tend to dilute or concentrate the solute.

Most published descriptions of changes in water quality during seepage relate the transport of solutes q to an average seepage velocity v and to molecular diffusion with an equation of the form

$$q = vC - D\frac{\partial C}{\partial x}, \tag{1}$$

where C is the concentration of solute, D an apparent diffusion coefficient, and x the distance. The unilateral use of equation (1) for all seepage velocities is plagued with trouble because neither term on the right exactly describes transport phenomenon. The actual velocity distribution within pores and pore sequences is not constant, nor can it be described as an explicit function. Moreover, the concentration of solute varies within pores and pore sequences, giving rise to an additional uncertainty when the term vC is used to describe the solute moving with the water. The second term has a similar uncertainty stemming from spatial variations of both concentration and water velocity within pores. For typical rates of soil-water seepage, the roles of soil-water velocity and diffusion in solute transport are inseparable — a phenomenon often overlooked or not recognized. Regardless of the quantitative shortcomings of equation (1), it aids in understanding the leaching process. The curves in figure 1 are those calculated from that equation and the equation of continuity.

In reality, the soil solution interacts with the soil particle surfaces during seepage. This interaction is brought about by the negative charge, the difference in mineralogical composition, and the difference in size and shape of the soil particles. Water quality changes during seepage because various solutes react differently with these particle surfaces. Moreover, the various solutes differ in their mobility and influence water movement by creating electroosmotic retarding forces. The magnitude of these forces depends on the solute concentration and soil water content.

During the past several years, we learned how to take advantage of these interactions rather than to be perplexed by their presence. We now know that we can initiate desired changes of salinity within soil profiles by irrigation. Experiments being conducted on cation exchange are revealing how the efficiency of the exchange process can be altered by controlling seepage rates at various depths within the soil profile.

We learned that tagging an inorganic or organic solute with a radioactive isotope does not provide readymade answers relative to identifying changes in water quality during seepage. To understand the leaching process, it is essential to observe simultaneously the movement of more than one solute relative to soil water behavior.

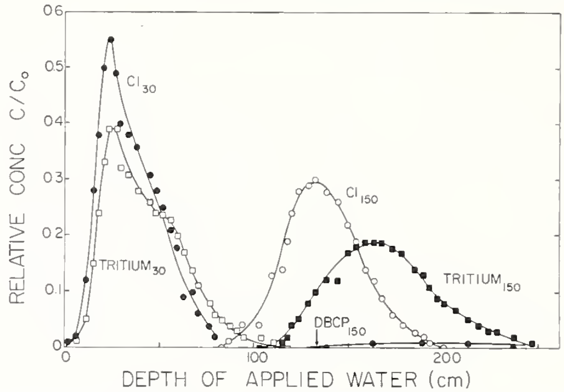

Figure 2.—Elution curves of chloride, tritium, and DBCP within a field soil measured at depths of 30 and 150 cm. during seepage. Source: Biggar, J. W., Miller, R. J., and Nielsen, D. R. Official data of Cooperative States Research Service Grant No. 716-15-5.

The data presented in figure 2 represent the concentration of chloride, tritiated water, and the nematocide, 1, 2-dibromo-3-chloropropane (DBCP) in the soil solution at the 30- and 150-cm. depths within field soil during seepage. The data stem from 15 cm. of irrigation water containing chloride, tritium, and DBCP at C_o concentration being leached through a nearly water-saturated soil with 250 cm. of solute-free irrigation water. Although the graphs resemble those in figure 1, several features are significantly different. Tritium is leached more slowly and never attains as great a relative concentration as that of chloride. Moreover, a comparison of the areas under the tritium and chloride curves reveals that some of the tritium is not being leached readily and remains in the soil profile. By the time the soil solution passes the 150-cm. depth, the DBCP concentration is less than 0.01, which indicates a much stronger interaction of DBCP with the soil particles than that of chloride or tritium. All solutes, regardless of their physical and chemical properties, will exhibit similar but characteristically unique behavior during seepage.

For a particular solute, equation (1) combined with the equation of continuity has been used to describe the change in water quality during seepage. A typical equation is

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} - v \frac{\partial C}{\partial x} \pm f_n \, (C, x, t); \; n = 1,2,3... \tag{2}$$

where C is concentration, t is time, and f_n are terms which account for losses or gains or both in solute concentration attributable to solution, precipitation, exchange, decomposition, sorption, decay, etc. Equation (2) does not accurately predict solute leaching in soils, but it is useful in analyzing the major processes that govern the solute behavior.

Our present ability to predict changes in water quality during seepage is inadequate. Work involving nonsteady flow conditions is both meager and incomplete. Investigations of water-quality changes within the crop root zone have yet to be reported. These will involve the wetting and drying of soil accompanied by reversals in water flow direction and characterized by a wide range of flow velocities. The translocation of organic solutes in soil is now being extensively studied due to an emphasis on soil and water pollution. Too often the organic solute movement and behavior are assumed, or they are measured under unreasonable seepage rates or uncontrolled and undefined leaching conditions. For the leaching process, insufficient emphasis has been given to the time-dependent nature of the composition of the soil air, the soil temperature distribution, the soil-water content distribution, and the nature and extent of the microflora.

More attention must be given to details,with a continued effort towards better methodology. Consider the common practice of equating chloride and nitrate distributions

within soils to evaluate mineralization, denitrification, or nitrate movement. Until recently, it had generally been assumed that chloride and nitrate (both having readily soluble salts and being univalent anions) would move identically in the soil in the absence of microbial activity. This assumption is not necessarily true. When water containing both chloride and nitrate was leached through several soils in Iowa, dif-

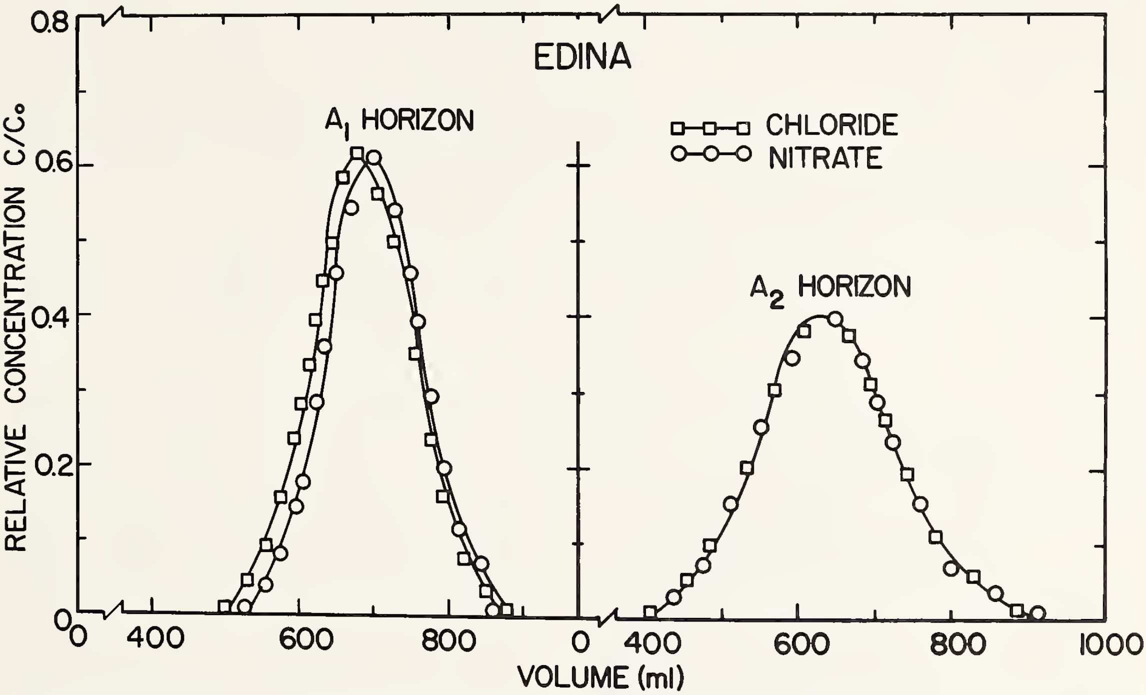

Figure 3.—Elution curves of chloride and nitrate from water-saturated soil columns. Source: Corey, and all (fig. 1).

ferences in chloride and nitrate behavior were clearly evident as shown in figure 3. For the A_1 horizon of the Edina soil, chloride moves faster than nitrate. For the A_2 horizon, chloride and nitrate movements are identical. These data and others showed that in the presence of organic matter, nitrate would not be displaced as easily as chloride. When microbial activity within these soils was eliminated by gamma radiation from a ^{60}Co source, the nitrate and chloride still moved at different rates. To study the movement of more complicated molecules, such as biodegradable solutes through soil, it will be necessary to determine the influence of microbial activity as well as the soil on the movement of the solute. This can be accomplished by conducting an experiment and then sterilizing the soil and repeating the experiment to determine the influence of soil alone on the movement of the solute. Radiation methods are ideally suited for this type of experimentation.

Without better methods for analyzing the decomposition and synthesis of soil organic compounds, including the degradation of surface-applied organic wastes percolating through soils, predictions of water quality during seepage will remain unreliable and incomplete. The fate of chemical materials dissolved in soil water must be determined from analyses of both the soil water and the soil air. The transient exchange of gases within the profile during seepage has received little attention. The classification of microbial activity into categories such as aerobic or anaerobic is untenable relative to the biological microenvironment during seepage. Gaseous concentration distributions are difficult to measure or control within soil profiles. Although not yet applied to field conditions, a method for monitoring changes in water quality and gaseous quality during seepage has been recently developed. Perforations along opposite sides of a cylinder wall containing a soil column being leached with water allow passage of a gas normal to the flow of water through an unsaturated soil. The water

within the soil pores, being at subatmospheric pressure and in thin films, allows a gas of known composition to diffuse readily and exchange with that dissolved in the soil water. The soil air may be continuously displaced by the gas or recirculated. The composition of the gaseous effluent may be monitored by gas chromatographic techniques while constituents dissolved in the liquid effluent may be analyzed by chemical or radioassay procedures.

Figure 4 depicts the fate of a solution containing chloride, nitrate, and sucrose at initial concentrations of C_0 as it is leached through a soil column 140 cm. long in contact with a continuous gas phase of 100 percent helium (anaerobic conditions). The nitrate elution curve indicates a recovery of only 73 percent of that added. The asymetrical nature of the nitrate curve and the large CO_2 concentrations indicate that the microbial activity increased markedly during leaching. Only 1 percent of the added

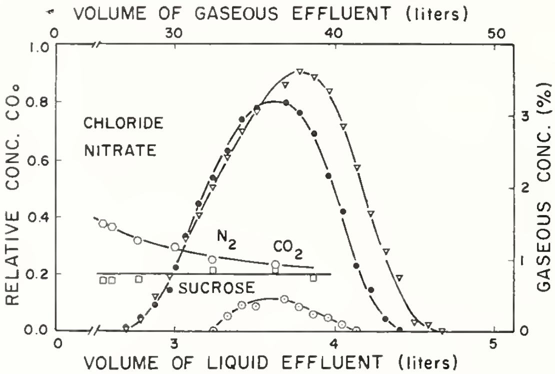

Figure 4.—Elution curves for chloride, nitrate, and sucrose from an unsaturated soil column during seepage. Values of N_2 and CO_2 were measured in the gaseous effluent created by passing 100 percent helium through the continuous gas phase of the unsaturated soil. Source: Mansell, R. S., Nielsen, D. R. and Kirkham, D. A method for simultaneous control of aeration and unsaturated water movement in laboratory soil columns. Soil Sci. (In press.)

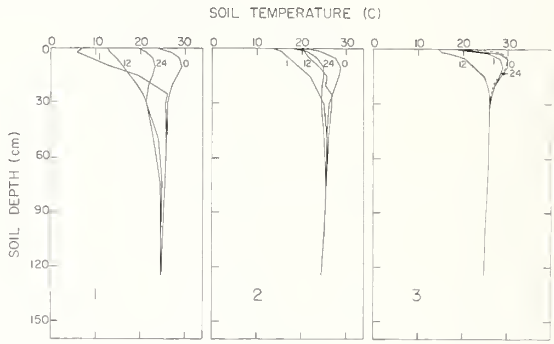

Figure 5.—Soil temperature distributions within a field soil for various times: (1) After the infiltration of 13.7 cm. of water at 2.6 C.; (2) After the infiltration of 13.7 cm. of water at 21.8 C.; and (3) With no irrigation. Source: Wierenga, P. J. An analysis of temperature behavior in irrigated soil profiles. [1968.] (Unpublished Ph.D. dissertation, Univ. Calif., Davis, Calif.)

sucrose was recovered in the liquid effluent. Because N_2 was found in the gaseous effluent initially free of N_2, denitrification was confirmed. This column technique is adaptable to many specific experiments dealing with filtration in the production of potable water and in the changes in water quality regardless of usage.

A review of water-quality changes during seepage would be incomplete without mentioning water temperature. Although we realize that a favorable temperature regime is paramount to all life, comparatively little work has been conducted on water temperature changes during seepage. Previous studies have included such practices as irrigation for frost protection or for lowering maximum daytime temperatures for seed germination. With increased water resource development and utilization, greater attention will be given to relatively small changes in soil water temperature and their influence on biological activity and attendant changes in both organic and inorganic solutes. Moreover, future sources of abnormally cold or hot water to be infiltrated into the soil mantle should be used to advantage by knowing the extent to which the soil microclimate may be altered.

How the temperature distribution within a soil profile changes during water infiltration is difficult to predict. Moreover, predicting the behavior of the water temperature following infiltration challenges even the best theoreticians.

An example of soil temperature behavior during and following infiltration is given in the three graphs of figure 5. For each graph, the soil, the initial soil-water content, and the initial soil-temperature distribution were virtually identical. The first graph depicts soil temperature distribution resulting from a 13.7-cm. irrigation with water at

2.6 C. The second graph gives temperature behavior for the 13.7-cm. irrigation with water at 21.8 C. For the third graph, no water was added. Zero time corresponds to 1800 Pacific Standard Time (PST) and to the time infiltration began. A 13.7-cm. irrigation infiltrated the soil in just less than 1 hour.

In graph 1, although the soil surface at 1 hour manifests the low temperature of the irrigation water, it is already beginning to warm up. At 12 hours, the temperature at the 50-cm. depth has been reduced 2 C. At 24 hours, the temperature has decreased 0.4 C at the 100-cm. depth. In graph 2, the soil surface at 1 hour is 8 C cooler than the temperature of the irrigation water. After 24 hours, the soil temperature remains unchanged below a depth of 80 cm. A comparison of graphs 1 and 2 with 3 reveals that irrigating the soil created differences in temperature behavior lasting more than 24 hours.

Such data collected in 10-minute intervals for a variety of initial soil and air temperatures, soil-water contents, irrigation frequencies, and irrigation water temperatures are presently being analyzed by diffusion models to predict water quality changes in relation to irrigation management. The extent to which soil temperature can be advantageously controlled under field conditions for creating desired changes in water quality remains undefined and is now holding the attention of many scientists.

Research efforts in the future will significantly enhance our capability of predicting changes in water quality during seepage. We will know how to relate the hydraulic properties of soils to the spread of inorganic solutes. The fate of gases mixing with the soil, air, and water will receive increased attention. The integration of biological activity with the more common physical and chemical soil properties is a necessity. The greatest utility of solute-free waters from atomic reactors or solute-laden waters stemming from man's activity remains to be found. And, in general, the adjectives "fresh" and "polluted" used to describe water quality will become even more ambiguous.

SEEPAGE AND SEEPAGE CONTROL PROBLEMS IN SANITARY LANDFILLS[1]

Frank R. Dair[2]

The subject of ground water pollution due to solid waste landfills is one that has had much investigation during the past two decades—especially in southern California. It is not surprising that this work should be spearheaded in southern California as few, if any, areas in the country are more reliant on ground water for water supply while, at the same time, depending almost entirely on landfilling as a means of solid waste disposal. Two separate research teams have been responsible for most of the work that has been done in this field in southern California. The University of Southern California (U.S.C.) has been almost continuously engaged in research on solid waste landfills since 1951, and during this period of time, various projects have been carried out under the direction of Professor Robert Merz, the principal investigator. U.S.C.'s work has been financed partly by the State Water Quality Control Board and partly by the U. S. Public Health Service (USPHS).

The other major investigations have been conducted by Engineering Science, Inc., under the sponsorship of the State Water Quality Control Board, and the USPHS Office of Solid Wastes Programs. The Sanitation Districts of Los Angeles County have cooperated with both groups in providing sites, constructing test cells, and providing other support.

Collectively, these studies have developed volumes of information. At the same time, however, they have produced few answers as to better methods of constructing or operating landfills so as to minimize or eliminate the hazards of ground water pollution.

These various studies show that some refuse landfills have the potential to degrade the nearby ground water. As a result of this information, the present policy of the California Regional Water Quality Control Boards and the agencies that advise them appears to be to deny the deposition of decomposable refuse in any location where there are underground aquifers separated from the refuse by only pervious strata such as sand and gravel.

From the standpoint of ground water management, this is, of course, a perfect solution. From the standpoints of both solid waste management and land management, however, this is a less-than-satisfactory answer.

In Los Angeles County alone, there are sand and gravel pits that either have been or will be excavated sufficiently to contain at least 200 million tons of refuse. In fact, if they can be used for this purpose, these pits will provide their tributary areas with an effective and economical means of refuse disposal for about 50 years. As compared with presently known alternate methods of disposal such as incineration, these pits represent a potential saving to the public of approximately a billion dollars.

From the land-management viewpoint, the abandoned excavations should be filled, as almost everyone will agree that an abandoned gravel pit is certainly not an aesthetic asset to a community. On the other hand, a pit once filled with refuse can be converted into a park or recreation area that will enhance its neighborhood, as well as provide a permanent open space.

The point of this discussion is that, although much research and investigation has been done, the work has not yet been carried far enough. Although there is much infor-

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[1] Contribution from the County Sanitation Districts of Los Angeles County, Los Angeles, Calif.

[2] Division engineer, Solid Waste Disposal Division.

mation regarding the pollutional effects of refuse fills, the data are mostly qualitative. More quantitative work is needed so that an intelligent trade-off can be made of the bad effects of each landfill on ground water versus the benefits. Also needed is research into methods of landfill construction that will eliminate or minimize the ground water degradation. A little work has been done along these lines, but much more is needed.

To discuss the effect of solid waste landfills on ground water, the discussion must be divided into three parts. The first case in which a refuse fill might affect ground water quality occurs where the wastes are deposited at such a depth as to be in direct contact with the ground water, either continuously, or intermittently, as the water level rises and falls. Some of Merz's earliest work in the early 1950's considered such a fill in Riverside, Calif., where the quality of ground water in the immediate vicinity of the fill was substantially degraded. One-half mile downstream from the fill, the ground water showed a noticeable increase in hardness, although it was still acceptable as a domestic supply. In California for the last 15 years, placing of decomposable refuse below the historic high ground water level is generally considered poor practice. Pits excavated lower than this must be filled with inert material to a level several feet above this historic high.

An interesting contrast to the situation in California is a landfill in another state that began operating in 1960 and knowingly placed refuse below ground water level.

The second case involves the leaching of a landfill by percolating water from either rainfall or irrigation that subsequently travels further downward to the ground water. The most recent project of U.S.C. placed heavy emphasis on this factor. If water is passed through household refuse, it becomes grossly polluted. Dissolved solids over 10,000 p.p.m. have been obtained, B.O.D.'s many times as strong as sewage have been measured and undesirable colors and odors have resulted. Refuse leachates are characteristically high in calcium and bicarbonate.

The question that arises, however, is whether water actually percolates through a fill in practice. Great amounts of applied water are needed to extract leachate from a test bin filled with refuse.

In the U.S.C. project, two test cells were constructed at the Sanitation Districts' Spadra Landfill to study the effects of rainfall and irrigation on a landfill. These cells were about 50 feet square on the bottom and larger on the top due to sloping sides. The overall depth of each cell was 21 feet, of which 19 feet was filled with typical household refuse, while the last 2 feet consisted of an earth cap or cover. The original intent of this experiment was to duplicate the rainfall in the Seattle area on one cell, while the other was to be planted with turf and irrigated sufficiently to maintain this turf. The irrigation system in this second cell was controlled by moisture-sensing devices located at the root zone. Each of the two cells was fitted with leach-collecting devices at various levels within the fill consisting of half-sections of 55-gallon drums.

The significant findings of this test insofar as the present discussion is concerned are that both of these cells received large amounts of water during the first 30 months, but no leachate was collected in any of the drums in the "heavy rainfall" cell, while only a small amount was collected in the uppermost collection drum of the "turf growing" cell. During the last 12 months, leachate in the drums was not recorded. In 42 months, the rainfall cell had received 170 inches of water (including an inadvertent heavy flooding), while the turf cell received 237 inches. Core samples of the cells were taken in February and November 1967 and the moisture content of the samples was compared with that of the original refuse. The moisture content of the rainfall cell decreased in the upper and bottom layers, while increasing in the middle layers for both the February and November core samplings. The turf cell's moisture content increased in all layers for the February core samplings and showed decreases in the upper layers and increases in the middle and lower layers in the November core samplings.

The Sanitation Districts have placed experimental underdrains under two large fills at the Mission Canyon Landfill to entrap leachate. So far, only one has yielded leachate, while the other, only malodorous gases. This water is apparently taking a short circuit path between the refuse fill and existing natural ground. This is evidenced by cracks due to settlement at these locations through which runoff rainwater enters. There is, in-

cidentally, no odor at the surface of the fill which attests to the efficiency of the earth-cover material in filtering them out. At the Mission Canyon Landfill approximately 20 inches of rainfall was once received in little more than a week and yet, upon excavation, the refuse was found to be wet for a depth of only 6 feet. During this same period, the fill did produce some leachate, but it emerged high up on the slopes, rather than at the base of fill. This water was apparently taking a short path from benches on the slopes to the sloping faces themselves. Interestingly, this leachate left rusty stains wherever it ran, which is believed to have been caused by leaching through a soil cover high in both ferric and ferrous oxides. This leaching was strictly a near-surface phenomenon and not a ground water problem. Concentrations of iron in leachate varied widely at different Sanitation District landfills in Los Angeles County, again indicating that it was picked up from soil cover, rather than from refuse. A more impervious covering on the benches probably is needed. These tests indicate that leachate is not a serious problem in a properly constructed landfill with controlled irrigation.

The third and most important point of concern regarding the effects of solid waste landfills on ground water is the possibility of gas migrating from the fill to the ground water. Extensive data accumulated on the composition of gases produced within a landfill show that the moisture content of the refuse exerts a determining influence on the gas produced. In general, however, a fill will produce methane and carbon dioxide. Engineering Science, Inc., found at a test site in Azusa that carbon dioxide will migrate for great distances vertically, laterally, and downward from a fill. The concern for ground water in this event arises from the likelihood that the CO_2 in contact with the ground water will dissolve lime from the soil and increase the hardness of the water.

Other work by Engineering Science, Inc., was aimed at developing barriers or membranes that could be placed under a fill to preclude downward and lateral travel. After some laboratory testing of materials, two test pits were installed at the Sanitation District's Calabasas Landfill. A polyethylene liner was tried, but failed drastically on the sides of the cell as it was filled with refuse. Another liner consisted of jute matting impregnated with a mixture of sand and asphalt. This material markedly reduced the flow of gas through the bottom, but results on the sides were not so favorable.

With the many new materials developed each year by the plastics industry, a test of only two types of liner could be considered only a beginning. Also worthy of study would be some system for ventilating the fill at the interface with the natural ground so that the gases could be exhausted, rather than traveling downward.

In summary, although much information has been gathered on the effects of refuse fills on ground water quality, much more work needs to be done. This work should be aimed both at producing more quantitative data and at developing better techniques of landfill construction so as to minimize the hazards of ground water degradation.

Fortunately, work is proceeding in both of these areas. U.S.C. is continuing their work at the Sanitation Districts' Spadra Landfill, part of which is devoted to a quantitative gas production study of refuse encased in a large steel tank. Simultaneously, the County Engineer of Los Angeles County has embarked on an extensive 3 year research program on landfills under a grant from the USPHS. This program is being conducted in conjunction with Engineering Science, Inc., and part of the project will be to continue the development of gas barriers for landfill sites located in pervious soils.

RECHARGE FROM INDUCED STREAMBED INFILTRATION UNDER VARYING GROUND-WATER-LEVEL CONDITIONS[1]

W. C. Walton[2]

INTRODUCTION

Man has augmented the natural infiltration of surface water into aquifers at many places by pumping wells or collectors near streams. Heavy withdrawals of ground water from well fields adjacent to streams lower the water table beneath streambeds and induce surface water to enter aquifers at high rates. Induced infiltration supplied by a perennial stream assures a continuing water supply even though overdraft conditions may exist in nearby areas supplied only by natural recharge from precipitation.

An aquifer-stream system in the Venice area, Ohio, for which hydrogeologic data are available was studied to gain insight into the magnitude of water-level declines

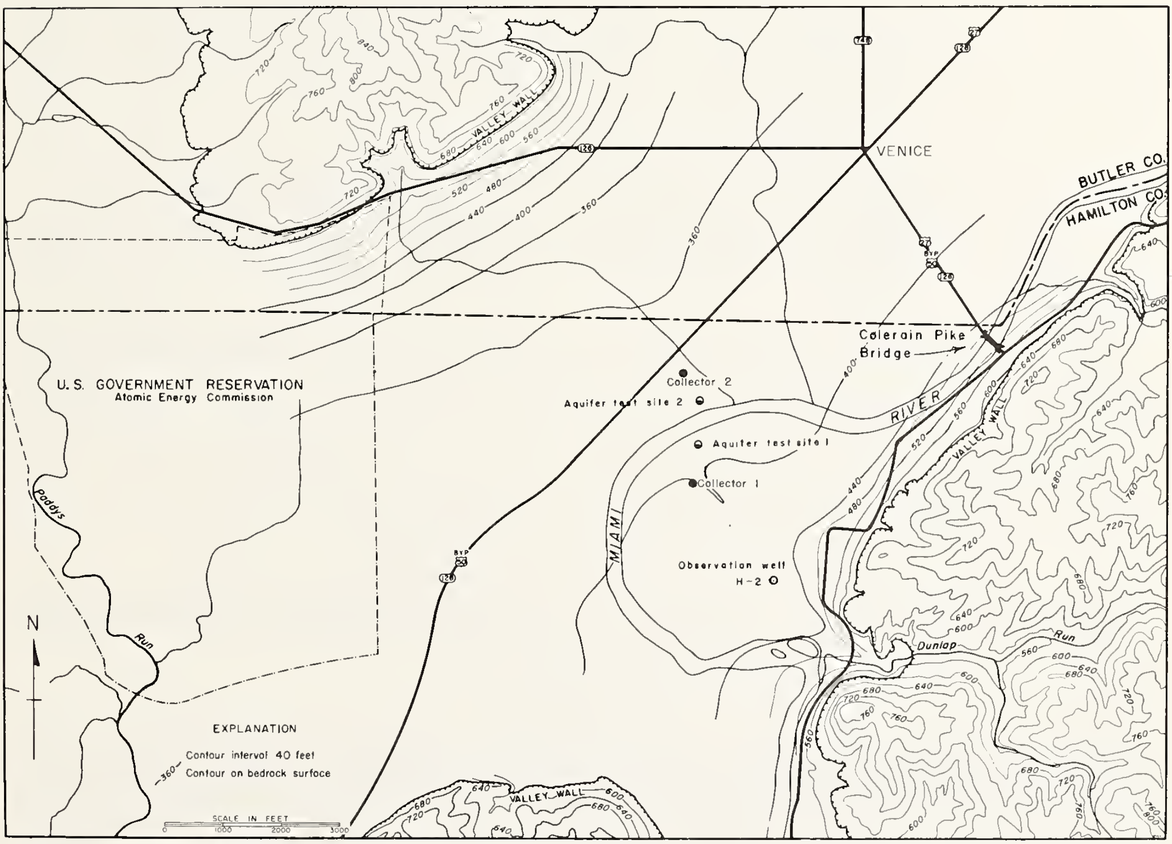

Figure 1.—Features of the Venice area, Ohio. Source: Dove, G. D. (see reference cited in footnote 4).

[1] The work upon which this publication is based was supported by funds provided by the U. S. Department of the Interior as authorized under the Water Resources Research Act of 1964, Public Law 88-379.

[2] Director, Water Resources Research Center, and professor of geology & geophysics, University of Minnesota, St. Paul, Minn.

under induced streambed infiltration conditions.[3] Venice is in southwestern Ohio about 12 miles northwest of Cincinnati and 7 miles southwest of Hamilton. The major source of ground water in the Venice area is the glacial outwash sand and gravel in the Miami River Valley. These outwash deposits are principally recharged by induced infiltration of surface water from the Miami River.

The Miami Valley is more than 2 miles wide in the Venice area as shown in figure 1. The valley represents the former course of a much larger stream. The stream cut its channel into the underlying consolidated rocks to a level more than 150 feet below that of the Miami River.[4] The valley has been filled to its present level chiefly with glacial outwash sand and gravel deposits into which the Miami River has cut its channel. The consolidated rocks, which occur beneath the outwash deposits and are exposed in the uplands, consist of relatively impermeable shale and interbedded limestone of Ordovician age.

The average precipitation at Venice is about 39.5 inches; the mean annual air temperature is about 56 F. A summary of streamflow records for the Miami River at Hamilton about 8 miles north of Venice follows: maximum daily discharge, 73,900 cubic feet per second (c.f.s.), Jan. 22, 1959; minimum daily discharge, 155 c.f.s., Sept. 27, 1941; mean discharge (Oct. 30, 1931 to Sept. 30, 1960), 321 c.f.s.; maximum recorded discharge, 108,000 c.f.s., Jan. 21, 1959; and minimum recorded discharge, 100 c.f.s., Sept. 26, 27, 1941.

The saturated thickness of the outwash sand gravel deposits in the Venice area averages about 125 feet and ranges in thickness from a feather edge near the valley walls to more than 220 feet near the center of the buried valley. The outwash deposits extend up and down the valley beyond the area which might be affected by heavy ground water withdrawals in the Venice area.

Aquifer tests were made in 1950 and 1954 at two sites to determine the hydrogeologic properties of the outwash deposits. The results of these aquifer tests indicate:

(1) The coefficients of transmissibility of the outwash deposits at sites 1 and 2 are 285,000 and 345,000 gallons per day per foot (g.p.d./ft.), respectively. The coefficients of permeability of the outwash deposits at sites 1 and 2 are 2,380 and 2,760 g.p.d./sq.ft., respectively.

(2) The coefficient of storage of the outwash deposits is in the water-table range and averages about 0.20. The average infiltration rate of the Miami River streambed in the Venice area was determined to be 168,000 g.p.d./acre/ft. based on water-table map and pumpage data.

Since 1952, the Southwestern Ohio Water Company has pumped large quantities of ground water from two collectors. The water is piped to manufacturing plants in the heavily industrialized Mill Creek valley north of Cincinnati.

ELECTRIC ANALOG COMPUTERS

Electric analog computers based on the aquifer-stream system in the Venice, Ohio, area were constructed. The analog computers consist of analog models and excitation-response apparatus (see reference listed in footnote 3).

Analog Computer 1

Analog model 1 is a regular array of resistors and capacitors. In the resistor-capacitor network, resistors are inversely proportional to the coefficients of transmissibility of the outwash deposits and the infiltration rate of the Miami River streambed; capacitors store electrostatic energy in a manner analogous to the storage of water within the outwash deposits. The electrical network is a scaled-down version of the Venice, Ohio, aquifer-stream system, assuming that leakage through the Miami River streambed is

[3] Walton, W. C., Hills, D. L., and Grundeen, G. M. Recharge from induced streambed infiltration under varying ground-water-level and stream-stage conditions. Minnesota Water Resources Res. Center, Bul. 6, 1967.

[4] Dove, G. D. A hydrologic study of the valley-fill deposits in the Venice area, Ohio. Ohio Div. Water, Tech. Rpt. No. 4, 1961.

directly proportional to declines in water levels beneath the streambed regardless of the stage of the water table.

Analog model 1 consists of a regular array of 1,985 resistors and 1,073 capacitors. The values of resistors range from 15 to 10,000 ohms and the values of capacitors range from 100 to 4,700 micromicrofarads. The fine grid of the model is 20 by 20 inches and has a scale of 1″ = 200 feet; the course grids of the model have scales of 1″ = 400 feet, 1″ = 800 feet, and 1″ = 8,000 feet. Scale factors[5] are $K_1 = 9.95 \times 10^{13}$ gal./coulomb, $K_2 = 1$ ft./volt, $K_3 = 2.77 \times 10^{7}$ g.p.d./a. and $K_4 = 7.30 \times 10^{6}$ days/sec. The analog model was constructed with ⅛-inch pegboard perforated with holes in a 1-inch square pattern. Aluminum angles (1 x 1 inch) were attached along the edges of the models with metal screws. Shoe eyelets were inserted in the holes of the pegboard to provide terminals for resistors and capacitors. A horizontal array of resistors and capacitors simulates the outwash deposits and a vertical array of resistors simulates the Miami River streambed.

Excitation-response apparatus forces electrical energy in the proper time phase into analog model 1 and measures energy levels within the energy-dissipative resistor-capacitor network. The excitation-response apparatus consists of two power supplies, a waveform generator, pulse generator, two power amplifiers and an oscilloscope as shown in figure 2.

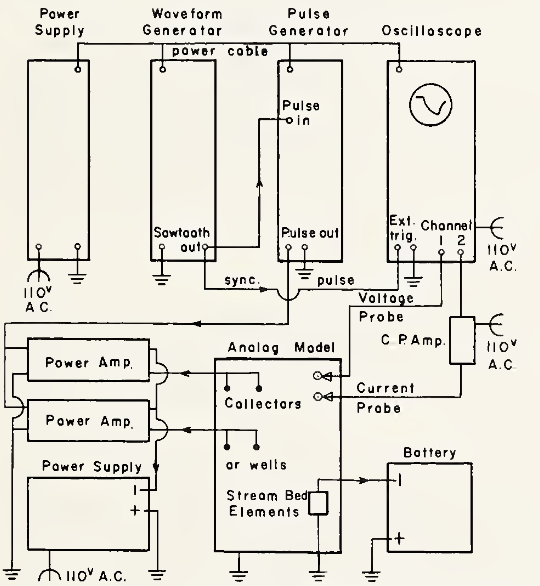

Figure 2.—Block diagram describing excitation-response apparatus. Source: Walton and others (see reference cited in footnote 3).

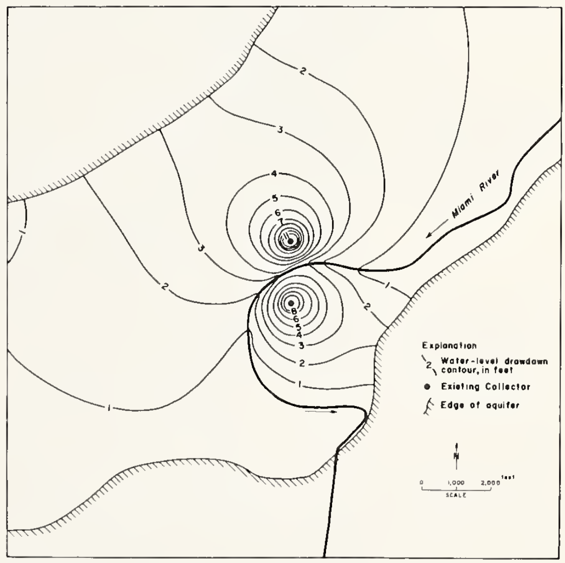

Figure 3.—Drawdown contours for two-collector system based on analog model 1. Source: Walton and others (see reference cited in footnote 3).

Analog model 1 was coupled to the excitation-response apparatus and the power amplifiers were connected to junctions at the locations of existing and hypothetical collectors or production wells. The output of the pulse generator was adjusted in accordance with selected pumping rates and periods. The oscilloscope was connected to terminals representing observation wells and water-level declines were computed from time-voltage graphs.

Drawdown contours for a two-collector system are shown in figure 3. Pumping rates of collectors 1 and 2 are 6.2 and 8.5 m.g.d., respectively. The drawdown contours are for a pumping period of 6 months. Thus, figure 3 shows the distribution of water-level decline as the result of pumping two collectors at constant rates for a period of 6 months with no recharge directly from precipitation but with recharge from induced streambed

[5] Walton, W. C. and E. A. Ackroyd. Effects of induced streambed infiltration on water levels in wells during aquifer tests. Minn. Water Resources Res. Center Bul. 2, 1966.

infiltration. Drawdown was measurable to distances of 32,000 and 33,000 feet up and down the buried valley from collector 2. Water-level declines beneath the streambed are less than 4 feet.

The minimum average depth of water in the Miami River during an extended dry period in August 1956 was about 4 feet. If it is assumed that stream stages remained at levels recorded in August 1956 during the selected 6-month pumping period and the water table before pumping started was level with the surface of the stream, then the water table was not lowered below the streambed and infiltration did not exceed maximum rates. Although analog model 1 was constructed by assuming that leakage through the streambed is directly proportional to drawdown beneath the streambed regardless of the stage of the water table, drawdown contours in figure 3 describe actual field conditions. However, drawdown contours would not be valid if the water-level declines beneath the streambed had exceeded 4 feet. Water-level declines in figure 3 are, therefore, approximately those that were observed in the Venice, Ohio, area during the summer and late fall of 1956. Except in short reaches of the Miami River where the stream was shallow and narrow, the water table is for August 1956 everywhere above the streambed.

Drawdown contours for a hypothetical eight-collector or well system are shown in figure 4. Assumed pumping rates for collectors or wells are given in table 1. The drawdown contours are for a pumping period of 6 months. Drawdown was measurable to distances of 50,000 and 44,000 feet up and down the buried valley from collector 2.

In this case, water-level declines beneath the streambed exceeded 4 feet along a 3,200-foot reach of the Miami River in the immediate vicinity of the well or collector field. Assuming the average depth of water in the Miami River was 4 feet during the selected pumping period and the water table before pumping started was level with the surface of the stream, the water table was lowered below the streambed in this reach of the river. The streambed elements were designed so that leakage of water through the streambed is directly proportional to drawdowns beneath the streambed regardless of the stage of the water table. Thus, recharge from streamflow in the immediate vicinity of the collector or well field exceeds the maximum infiltration rate which is reached when drawdowns are 4 feet. If maximum infiltration conditions were considered, drawdowns associated with the eight-collector or well field would be greater than those shown in figure 4. Analog model 1 with a vertical array of resistors simulating the streambed is not valid for the selected high rate of pumping and actual stream-stage and water-table conditions.

TABLE 1. — Assumed pumping rates for an 8-collector or well system in Venice area, Ohio[1]

Collector or well No.	Pumping rate (m.g.d.)
1	4.9
2	3.2
3	4.7
4	4.8
5	4.2
6	4.8
7	2.9
8	3.5
Total	33.0

[1] From Walton and others (reference listed in footnote 3).

Analog Computer 2

Analog model 2 is the same as analog model 1 except that the Miami River streambed simulation is different. In analog model 2, maximum induced infiltration is taken into consideration by simulating the streambed with vertical arrays of resistors, transistors,

and diodes instead of simple vertical arrays of resistors as in analog model 1. The streambed elements were designed so that leakage through the streambed reaches a maximum rate when the water-level decline is 4 feet and thereafter remains constant. Therefore, analog model 2 is valid for any pumping rate, whereas analog model 1 is valid only for pumping rates low enough so that the water table is not lowered below the streambed.

The elements simulating the streambed in analog model 2 were designed for the following conditions: 1) the current through a streambed element must increase directly proportional to the associated aquifer element voltage drop until the voltage is 4 volts (drawdown beneath the streambed is 4 feet); 2) thereafter the current through a streambed element and the voltage drop across the element must remain constant; and 3) the aquifer elements must at all times be affected by current moving through the streambed elements, but the voltage of the aquifer elements must continuously drop after the current through the streambed element becomes constant. Streambed elements were assembled and attached to the streambed-aquifer nodes of the analog model. Analog model 2 was coupled to the excitation-response apparatus and the power amplifiers were connected to junctions at the locations of the eight hypothetical collectors or wells shown in figure 5. The output of the pulse generator was adjusted in accordance with the assumed pumping rates in table 1 and a pumping period of 6 months.

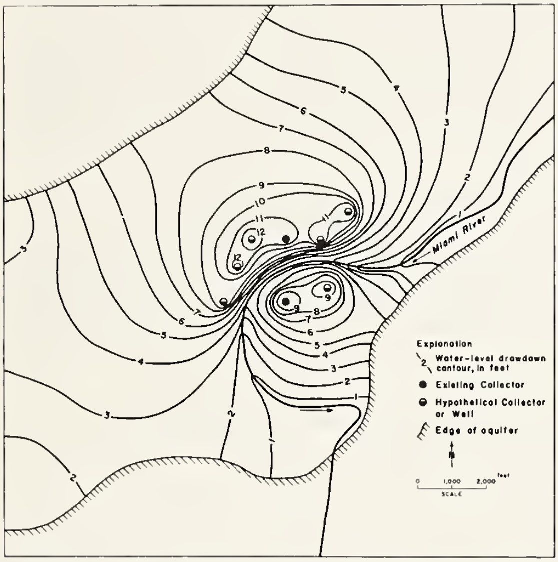

Figure 4.—Drawdown contours for eight-collector or well system based on analog model 1. Source: Walton and others (see reference cited in footnote 3).

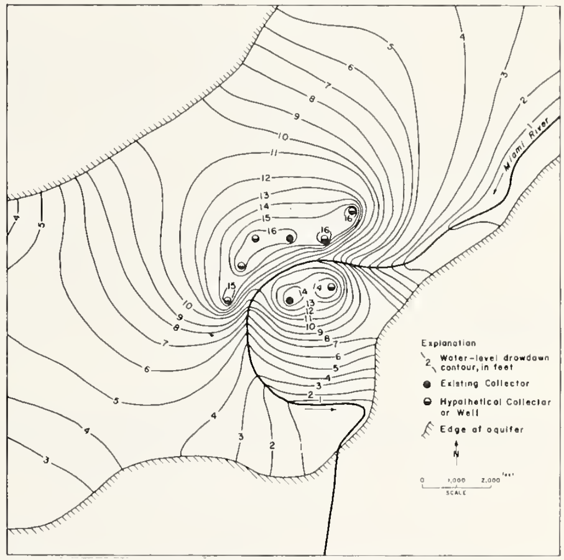

Figure 5.—Drawdown contours for eight-collector or well system based on analog model 2. Source: Walton and others (see reference cited in footnote 3).

Drawdown contours for the eight-collector or well system are shown in figure 5. Drawdown was measurable to distances of 50,000 and 44,000 feet up and down the buried valley from collector 2. A comparison of figures 4 and 5 shows that much greater drawdowns occur when maximum infiltration conditions are considered rather than ignored. With the streambed correctly simulated, water-level declines beneath the streambed exceed 4 feet along a 5,500-foot reach of the Miami River (fig. 5), whereas with the streambed incorrectly simulated, water-level declines beneath the streambed exceed 4 feet along only a 3,200-foot reach of the Miami River. With the streambed correctly simulated, water-level declines near the collectors or wells north and south of the river average about 16 and 14 feet, respectively, whereas with the streambed incorrectly simulated, water-level declines near the collectors or wells north and south of the river average about 11 and 9 feet, respectively. The importance of correctly simulating the streambed is apparent.

CONCLUSIONS

In the past, little, if any, attention has been given to the complex nature of recharge from induced streambed infiltration under varying ground-water-level conditions. Streambeds are often simulated in mathematical models by hypothetical recharge boundaries located with aquifer-test data. Under heavy pumping, the recharge boundaries may not correctly simulate leakage through streambeds and computed drawdowns may be greatly in error.

Streambeds have been simulated in analog models by a ground wire which is equivalent to simulating a streambed as a hypothetical recharge boundary. In some analog models, the streambed is simulated with vertical arrays of resistors. Simulation of induced infiltration by a hypothetical recharge boundary, such as a ground wire or simple vertical arrays of resistors has been shown to be incorrect for certain conditions, and will lead to misleading estimates of the potential yields of aquifer unless the pumping conditions are such that the water table is not lowered below the streambed and maximum induced infiltration rates do not occur.

In evaluating ground-water resources, streambeds must be simulated in mathematical and analog models in such a way that the following conditions are considered:

1. Leakage of water through a streambed is directly proportional to the drawdown beneath the streambed until the water table declines below the streambed. Thereafter, induced infiltration remains constant provided the stream stage and temperature remains stationary.

2. Provided that the water table remains below the streambed, leakage of water through a streambed is directly proportional to the average depth of water in the stream and varies with stream-stage changes and changes in the temperature of the surface water.

WATER-QUALITY IMPROVEMENT BY GROUND-WATER RECHARGE[1]

Herman Bouwer[2]

INTRODUCTION

Seepage of low-quality water is often considered a source of ground-water pollution. However, the quality of seepage water may be improved considerably as it moves through the soil. Percolation of low-quality water through soil is an effective mechanism for removing biodegradable material, bacteria, viruses, and certain inorganic substances from the water. For this reason, "induced" seepage from surface inundations to ground water is being used increasingly as an economical tool for renovating and storing low-quality water, such as treated sewage and polluted river water. A significant part of waste water renovation by movement through soil takes place in connection with drainage from septic tanks and with seepage from sewage lagoons or polluted streams. This paper, however, will be limited to artificial installations with deliberate and controlled entry of low-quality water into the soil and subsequent percolation to and below the ground water. Because this seepage is the objective rather than the undesired loss of water, the phenomenon is no longer called seepage, but recharge.

REMOVAL OF ORGANIC AND INORGANIC COMPOUNDS

Polluted surface water may contain organic and inorganic compounds and living matter such as algae, insect larvae, and other macroorganisms, protozoa, bacteria, and viruses. When such water moves into and through soil, the large representatives of the living BOD and other suspended material remain on the soil surface. Metals such as iron, manganese, nickel, copper, zinc, lead, and cadmium may become immobilized at or near the surface of the soil if alkaline and aerobic conditions prevail[3].

In the first few feet of soil, intensive microbiological activity causes essentially complete removal of carbonaceous and nitrogenous oxygen demand (at least for secondary municipal effluent and similar wastes) with carbon dioxide, water, nitrates, sulphates, phosphates, and other minerals and stable organic compounds as end products (*2*, *5*, *6*, *7*). The biota involved is in delicate balance with the total environment and difficult to predict. Clark (*3*) in his chapter on the growth of bacteria in soil states, "Hopefully, some of the data presented (in the chapter) will be used as a basis for asking, but not for answering, why one or another species grows in a particular microsite in soil and why soil organisms, individually or collectively, often fail to exhibit growth responses of some expected magnitude in given situations." The biological "cleansing" ability of the soil is readily appreciated, however, if one realizes that the soil has been receiving and decomposing plant residues and animal wastes for ages. Also, the microorganisms responsible for biological purification of waste water in sewage treatment plants all have soil as their common origin.

Phosphates, present in the effluent or formed by the mineralization of proteins or detergents, are highly immobile in the soil and appear in low concentrations in the renovated water (*7*, *8*). Application of waste water on cropped areas in amounts not much in excess of the plant consumptive use may result in considerable removal of phosphorus, nitrogen, potassium, and other elements by the harvested parts of the crops (*7*).

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[1] Contribution from the Soil and Water Conservation Research Division, Agricultural Research Service, U. S. Department of Agriculture.

[2] Research hydraulic engineer, U. S. Water Conservation Laboratory, Phoenix, Ariz.

[3] Lehman, G. S. Soil and grass filtration of domestic sewage effluent for the removal of trace elements. 1968. (Unpublished Ph.D. dissertation, Univ. Ariz., Tucson, Ariz.)

REMOVAL OF BACTERIA

Downward movement of low-quality water in soil effectively removes fecal organisms. This removal takes place primarily in the first 5 to 10 ft. of the soil (*2, 5, 6*). Coliforms in samples obtained at depths of about 6 ft. or more below a recharge basin have been detected in several instances, but they were of soil rather than of fecal origin (*6*). Virus mortality is more difficult to determine but evidence is accumulating that movement of water through soil is also effective in removing virus.

At the Flushing Meadows project, a pilot facility to determine the feasibility of renovating secondary effluent from the Phoenix area by ground-water recharge through surface spreading,[4] the most probable number (MPN) of coliforms 30 ft. below the recharge basins (15 ft. below the ground-water table) for a "wet" schedule of inundations is about 20 per 100 ml. (presumptive test). These bacteria, however, are almost all of nonfecal origin, as determined by the Eijkman test on EC broth.

Although most of the micro-organisms are removed in the first 5 to 10 ft. of soil percolation, the unpredictable behavior of microbiological systems in the soil requires additional underground travel of the water to safeguard against the possibility of persistence of pathogenic organisms. Little is known regarding the desired distance or time of underground travel. As a guide, at least 500 to 1,000 ft. of horizontal travel below the water table and a detention time of several months should be allowed before the water is collected by drains or wells for reuse. These magnitudes also depend on the soil materials in question and on the intended use of the renovated water. Municipal use, for example, will demand more stringent safety precautions than irrigation or return of the water to lakes or streams. The travel below the water table following the downward percolation from the basins to the water table can also be effective in removing taste and odor from the water. Of course, the soil formations should not be so coarse textured that insufficient surface area is available for the biological and chemical reactions. "Short circuiting" should be avoided at all times. A good soil condition would be fine sands or sandy loams beneath the basins and sands and gravels in the aquifer.

SCHEDULING INUNDATIONS AND DRYUPS

In recharging with low-quality water, suspended material should be removed as much as possible before the water is admitted into the recharge basins. This can be achieved by passing the water through sedimentation basins followed by filtration through gravel or vegetation. Gravel filtration can be accomplished with separate filtering facilities, or the bottom of the recharge basin can be covered with a layer of relatively fine gravel. The latter has been used successfully in the Peoria, Ill., and the Los Angeles, Calif., recharge facilities (*6*). Vegetation filtration, which is effective in removing suspended material, oxygen-demanding wastes, and coliforms from the waste water (*9*), can be achieved by flowing the effluent through a dense stand of bermudagrass or other inundation-tolerant plants. The grass can be grown in the recharge basin itself. Such a basin should be long and narrow, like an irrigation border, to achieve sufficient distance of overland flow. The effect of a gravel cover and that of vegetation on the infiltration behavior of recharge basins will be studied at the Flushing Meadows project.

Use of low-quality water for ground-water recharge requires that predominantly aerobic conditions be maintained in the soil profile beneath the recharge basins. This is to avoid reduction and resulting mobilization of metals, and biological reactions that lead to incomplete digestion of the biodegradable materials or to clogging soil (such as formation of methane and polysaccarides). Also, bacteria of fecal origin tend to have a greater mortality in aerobic than in anaerobic soil environments.

To maintain aerobic conditions, inundation must be intermittent, so that atmospheric oxygen can enter the soil profile during drying cycles. The two mechanisms whereby oxygen can enter the profile are (1) diffusion caused by a concentration gradient of the

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[4] This is a cooperative study between the U. S. Water Conservation Laboratory and the Salt River Project; it is partly supported by a Demonstration Grant from the Federal Water Pollution Control Administration.

oxygen in the soil air, and (2) mass flow as air replaces the liquid that is draining from the profile due to gravity.

The length of the inundations and dryups of the basins should be selected so that enough oxygen enters the soil during dryups to satisfy the total oxygen demand of the water that enters the soil during inundations. This total oxygen demand can be estimated as the sum of the chemical oxygen demand and the oxygen necessary for oxidation of ammonium and organic nitrogen to nitrate (*6*). For secondary effluent, this total oxygen demand is about 150 to 200 milligrams per liter. Inundation schedules practiced have varied from about 8 hours wet and 16 hours dry (*6*) to 5 days wet and 10 days dry (*1*).

The schedule of inundations and dryups theoretically is controlled by the total oxygen demand of the effluent, the intake rate of the basins as determined by the hydraulic conductivity and clogging potential of the soil profile, the rate of drainage of the profile following a dryup, and the depth of the water table. Theoretical analysis of the intermittent inundation problem and a computation of desired inundation schedules in relation to effluent quality, soil hydraulic conductivity and porosity, and water table depth are currently underway at the U.S. Water Conservation Laboratory. In areas where mosquitoes or other insects are a problem, the inundation schedule would have to be adjusted so as to not coincide with the mosquito cycle.

The ground-water mound should not be permitted to rise higher than 5 to 10 ft. below the bottom of the recharge basins. This is particularly important where the recession of the mound is so slow that it will significantly restrict the drainage rate of the soil profile during dryups. Thus, a thorough knowledge of the soil hydraulic conductivity above the water table and of the hydraulic properties of the aquifer is required to determine the correct size and spacing of recharge basins, the permissible recharge rates, and the best location of the wells or drains for collecting the renovated water. The objective should be to obtain maximum recharge with a minimum of basin area while avoiding waterlogging of the soil beneath the basins.

The water table at the Flushing Meadows project is about 15 ft. deep. Two observation wells, 30 and 100 ft. deep, with nonperforated casing down to the bottom, were installed in the center of the 220- x 700-ft. basin area. The aquifer consists of sand and gravel layers to a depth of about 245 ft. where a clay deposit begins. Because of the succession of layers of different material, the aquifer can be expected to be anisotropic, with the hydraulic conductivity in horizontal direction exceeding the one in vertical direction.

Inundation of the basins caused the water levels in the observation wells to rise to pseudoequilibrium levels, which for a particular period during which the average infiltration rate was 3.5 ft. per day, were 2.7 and 0.8 ft. above static water level for the shallow and deep well, respectively. To determine the hydraulic conductivity components, a model of the aquifer was set up on a resistance network analog. The ratio of horizontal to vertical hydraulic conductivity in the analog model was then varied until the analog correctly simulated the observed water level rises in the shallow and the deep well. From the known infiltration rate and the "entry" current in the analog model, the hydraulic conductivity components were computed as 282 ft. per day in horizontal direction and 17.6 ft. per day in vertical direction. These high values are extremely favorable for ground-water recharge, since the absence of restriction to vertical movement and the very rapid lateral disposal of the water will allow large, closely spaced basins and high recharge rates. For the 220- x 700-ft. basin area of the Flushing Meadows project, the ground-water mound rose only 3.8 ft. for an intake rate of 3.5 ft. per day.

NITRATE REDUCTION

If municipal sewage is used for recharge, the total nitrogen content of the influent can be expected to be about 20 to 40 p.p.m. (higher for regions with a low per capita water use). After aerobic percolation through soil, this nitrogen will be essentially converted to nitrate. Because of the mobility of the nitrate ion, the nitrate content of the renovated water will thus also be in the range of 20 to 40 p.p.m. This is well above the limit of 10 p.p.m. for drinking water set forth by the U.S. Public Health Service. This high nitrate content may be of value where the renovated water is used for irrigation (20 to 30 p.p.m.

N corresponds to 54 to 81 lb. of N per acre-foot of water). There will be instances, however, when a high-nitrate content in the irrigation water is undesirable. High-nitrate contents will also be objectionable where the renovated water is released into streams or lakes because of possible eutrophication.

The high-nitrate content of renovated sewage effluent is presently the most serious problem in waste water renovation by ground-water recharge. It is prompting intensification of research in several parts of the world to determine how this nitrate content can be reduced. The simplest solution, of course, would be to dilute the renovated water with water of a much lower nitrate content. Other possibilities are raising the pH of the effluent for volatilization of ammonia, and recharge basin management to induce maximum denitrification. The latter is a microbiological process that occurs where nitrates and a biochemical oxygen demand are combined under anaerobic conditions. In that case, the oxygen of the nitrate ions is used by the bacteria in their metabolism of organic material, resulting in the formation and release of nitrogen and nitrogen oxide (*2*). To obtain denitrification, the dryup periods should be selected so that the amount of oxygen entering the soil is not sufficient to meet the total oxygen demand of the effluent. Thus, when the fluid reaches the water table where oxygen is no longer available, nitrates may have been formed while oxygen-demanding substances are still present.

For the Flushing Meadows project, the nitrate content of the renovated water at 30-ft. depth in the center of the basin area varied from 1-12 p.p.m. N, if a "wet" inundation schedule was practiced (5 days wet, 2 days dry, for example). However, if a "dry" schedule was maintained (2 days wet, 5 days dry), the nitrate content in the renovated water rose to 30-35 p.p.m. N. For both schedules, the ammonium content of the renovated water was about 2 p.p.m. N, the nitrite content was 0.4 p.p.m. N, and the organic nitrogen content was about 0.5 p.p.m. N. The lower nitrate content of the renovated water under a wet inundation schedule is attributed to denitrification.

MINIMIZING GROUND-WATER POLLUTION

Where the quality of the renovated water is inferior to that of the "native" ground water, the extent of groundwater pollution can be restricted by maintaining a gradient in the native ground water that is directed toward the area of recharge activity. This can be accomplished, for example, by placing a ring of wells or drains at the proper distance around the spreading area and collecting water from the wells or drains so that there is always some gradient in the "outside" ground water toward the wells or drains. With such a system, part of the aquifer is then actually zoned for recharge, enabling low-quality water to be renovated and stored while avoiding quality degradation of the native ground water in the rest of the aquifer.

ECONOMIC AND AESTHETIC ASPECTS

The cost of renovating waste water by ground-water recharge depends on the cost of the land and of constructing and operating the basins and distribution works, on the amount of water that can be infiltrated per unit time and per unit area, and on the cost of pumping or otherwise collecting the renovated water. Assuming that the amortization and operational costs of the recharge basins are $1,000 per acre per year, which is probably on the conservative side, and assuming a recharge rate of 200 ft. per yr., the cost of renovating an acre-foot of water would be $5. If unfavorable soil or aquifer conditions permit an annual recharge rate of only 50 ft. per yr., this figure would increase to $20. Thus, where feasible, spreading for ground-water recharge is an effective and economical method for renovating sewage effluent or other low-quality water (*4*).

An intangible but important advantage of waste water renovation by recharge is its aesthetics. When the sewage effluent percolates through the soil and moves below the water table, it loses its identity as sewage and when it reaches a well, it is pumped up as ground water. Such an invisible and "natural" process of waste water renovation will probably meet with greater public acceptance than purification by activated carbon adsorption or other methods of direct or "closed-loop" treatment, where the connotation of "purified sewage" tends to persist.

SUMMARY

Aerobic percolation of low-quality water through soil to the water table and subsequent lateral movement below the water table can be effective mechanisms for removing biodegradable material, pathogenic organisms, and certain inorganic substances such as metals from the water. Thus, induced seepage, or ground-water recharge, is increasingly used to renovate sewage effluent or other polluted waters.

Recharge basins should be managed to avoid accumulation of suspended material on the bottom and to allow sufficient oxygen to enter the soil during dryups to satisfy the total oxygen demand of the water that has entered during inundation. Also, the basins should be so laid out and operated to avoid excessive water-table buildups while at the same time achieving maximum recharge per unit basin area.

Nitrogen in the low-quality water will appear mostly as nitrates in the renovated water. For renovated sewage effluent, the nitrate content may be two to four times higher than the permissible limit for drinking water. Nitrate reduction by denitrification or other processes is emerging as a major problem in renovating sewage effluent, and research in this area is being intensified.

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DISCUSSION

SESSION I — SEEPAGE AND WATER QUALITY

R. Rice: What type of leachate sampling device was used to obtain water samples for quality analysis?

A. Grimm (presented paper for F. R. Dair): The leachate enters a perforated French drain that lies on the canyon floor and is covered by approximately 40 feet of refuse. The French drain is connected to a 12-inch steel pipe that was jacked through an earth barrier across the canyon mouth that was compacted in place prior to placing the refuse behind it. The steel pipe is set on grade to drain away from the fill and capped with a victaulic-end cap. The leachate is sampled through a ¼-inch valve tapped into the victaulic cap.

R. Rice: Do you get saturated or unsaturated flow in the infiltration processes?

A. Grimm: The experiment being carried out by the University of Southern California at the Sanitation Districts' Spadra Landfill shows that the flow in a landfill can be unsaturated.

In this test no liquid leachate was captured by collection pans placed in the test cell for that purpose and yet moisture did travel downward as indicated by an increase in moisture content in samples of refuse obtained from the lower levels by coring.

L. S. Willardson: Would development of ground water resource along the Miami River cause the river to dry up?

W. C. Walton: No — because the minimum daily flow in the Miami River is 155 c.f.s. whereas the maximum daily induced infiltration is about 40 c.f.s.

L. S. Willardson: Are there possibilities of downstream objections?

W. C. Walton: No — there are no water demands downstream.

A. Halderman: Please comment further on the point regarding metals immobilization. Does this occur under aerobic or anaerobic conditions and what is the effect on permeability?

H. Bouwer: The immobilization of certain metals takes place if alkaline and aerobic soil conditions prevail. Most studies on waste water renovation by soils percolation and ground-water recharge cover only a limited period of time and long-term behavior is not very well known. Whether the permeability of the surface soil would decrease due to immobilization of certain metals and other elements, or whether the efficiency of metal removal would go down, is a question that is difficult to answer at the present time. Under long-term operation, restoration or relocation of recharge basins may well become necessary.

W. A. Lidster: Is there a given rate of infiltration that will give optimum bacteriological cleansing? I'm thinking of seepage from a sewage lagoon located near a reservoir utilizing the water for municipal and industrial purposes along with irrigation.

H. Bouwer: In general, the lower the infiltration rate, the more complete the removal of bacteria that will be obtained. Dirty water, such as raw sewage or primary effluent, will require lower infiltration rates than secondary effluent or other cleaner waters.

In this respect, there is some self-adjusting mechanism at work, because anaerobic conditions and soil clogging, which will reduce the infiltration rates, occur much faster with low-quality water than with high-quality water. The main factors in obtaining bacteriologically safe renovated water, however, are probably time and distance of travel through granular or other soil formations without short circuiting.

SESSION II A. — HYDROLOGY AND SEEPAGE

Chairman: K. K. Barnes

THE EFFECT OF SEEPAGE LOSSES ON STREAM REGIMEN

Donald D. DelManzo Jr.[1]

INTRODUCTION

Verification by testing is a time-honored means by which an engineering hypothesis achieves professional acceptance. Analysis techniques have been formulated for estimating underground water movement in the form of return flows resulting from irrigation.[2] It is the purpose of this paper to apply these techniques to irrigation and precipitation water in a mathematical model of a large-scale "river-aquifer system." A river-aquifer system shall herein describe a valley containing a perennial stream underlain by alluvial, water-conducting deposits such that essentially unobstructed flow can occur between stream and aquifer, and vice-versa. These techniques are important to the analysis of the complex problem encompassed by a river-aquifer system, because they enable the analyst to quantify the underground flows accurately, an absolute necessity in the optimization of water usage.

PROTOTYPE SELECTION

The choice of a particular reach of a river-aquifer system on which to test the combined analysis techniques was based upon certain requirements. It was desirable that the stream have good gaging stations, that substantial data and observations be available for that reach, that there be no large tributary streams within the reach, and that the valley be representative of other river-aquifer systems. The section of the South Platte River between the gage at Kersey and the one at Julesburg, Colo., satisfies these requirements. This reach comprises approximately 140 miles of stream and 250,000 acres of irrigated land.[3]

ASSUMPTIONS AND IDEALIZATIONS

The assumptions and idealizations necessary in the construction of the model are:

1. A valley with a river flowing down the center and gaging stations at each end;

2. A permeable stratum of uniform width across the valley with uniform depth and uniform thickness;

3. The validity of Dupuit's assumption;

4. The areal irrigation applications and areal precipitation are uniform;

5. The consumptive use is uniform throughout the valley.

Additional assumptions, necessary to analyze certain particular terms in the budgetary equation, will be delineated in the discussion of each component.

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[1] Lieutenant, CEC, U. S. Navy; graduate student, Colorado State University, Fort Collins, Colo.

[2] Glover, R. E. Return flow from an irrigated strip. 1962. Unpublished data of the U. S. Bur. Reclam., Water Resources and Utilization Section; Hurley, P. A. Predicting return flow from irrigation. 1961. U. S. Bur. Reclam. Tech. Memo No. 660; Maasland, M. Water table fluctuations induced by irrigation. Amer. Soc. Civ. Engin. Proc. 87 (IR 2):30-58. 1961.

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PROCEDURE

To substantiate the validity of the ground-water analysis procedures, a model was developed to mathematically describe the interrelationships in the South Platte. The principle of the model was to use known input data, assumptions, idealizations, and analysis to calculate an estimated surface outflow at the downstream gage. If the calculated outflow significantly replicated the recorded outflow, then the theoretical procedures would be verified. The following budgetary equation was used: calculated flow at Julesburg = flow at Kersey + reservoir return flow + irrigation return flow + base return flow — diversions to storage — diversions to irrigation — change in channel storage. Each term was quantified by the methods described below.

Flow at Kersey

Records for the flow at Kersey were obtained from the U.S. Geological Survey.[4]

Irrigation Return Flow

Perhaps the most sensitive calculation within the model is that of the irrigation return flow, because it is highly dependent upon the assumptions, and because its effects are long term. Return flow estimates depend upon the choice of valley width, the aquifer constant, the position of the stream, the consumptive use coefficient, and the assumed pattern of irrigation application, each of which must be based upon the results of various field tests or estimates or both.

The idealization selected to approximate the actual pattern of water application will significantly influence the pattern of the return flows. Two basic assumptions have been used in idealizing the application of precipitation and irrigation water. The first was developed by Glover and was utilized extensively by Hurley in his work (see footnote 2). It is based upon the premise of an instantaneous application at the middle of each month. The second, developed and used by Maasland, assumes uniform continuous infiltration throughout the month (see footnote 2). The second method was chosen as a more realistic idealization of actual precipitation and irrigation.

Diversions to Storage

Records for the storage diversions along the river were obtained from the Colorado Office of the State Engineer.[5]

Diversions to Irrigation

Records for the irrigation diversions were obtained from the Northern Colorado Water Conservancy District.[6]

Reservoir Return Flow

Because of the existence of ground-water mounds as a result of leakage beneath several of the irrigation storage reservoirs, return flow to the river is continuous and constant, or very nearly so, throughout the year. Dille[7] estimated the losses to the river from the mound under each reservoir, and combined them into one constant.

Included within the irrigation return flow is the component resulting from the deep percolation of precipitation. To employ the Maasland method, a total volume of water that deep percolates to the water table must be calculated for each month. This total applied water is equal to the sum of the irrigation applications plus precipitation minus consumptive use. A part of this applied water returns to the stream each succeeding month.

Consumptive Use

Consumptive use incorporates all water losses to evapotranspiration, including those which result from recirculation of water by pumping. Munson's procedure for estimating the consumptive use of water for agriculture was adopted.[8]

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[4] U. S. Geological Survey. Water supply paper No. 1378. Washington, D. C. 1957.

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[6] Northern Colorado Water Conservancy District. Unpublished data on reservoirs.

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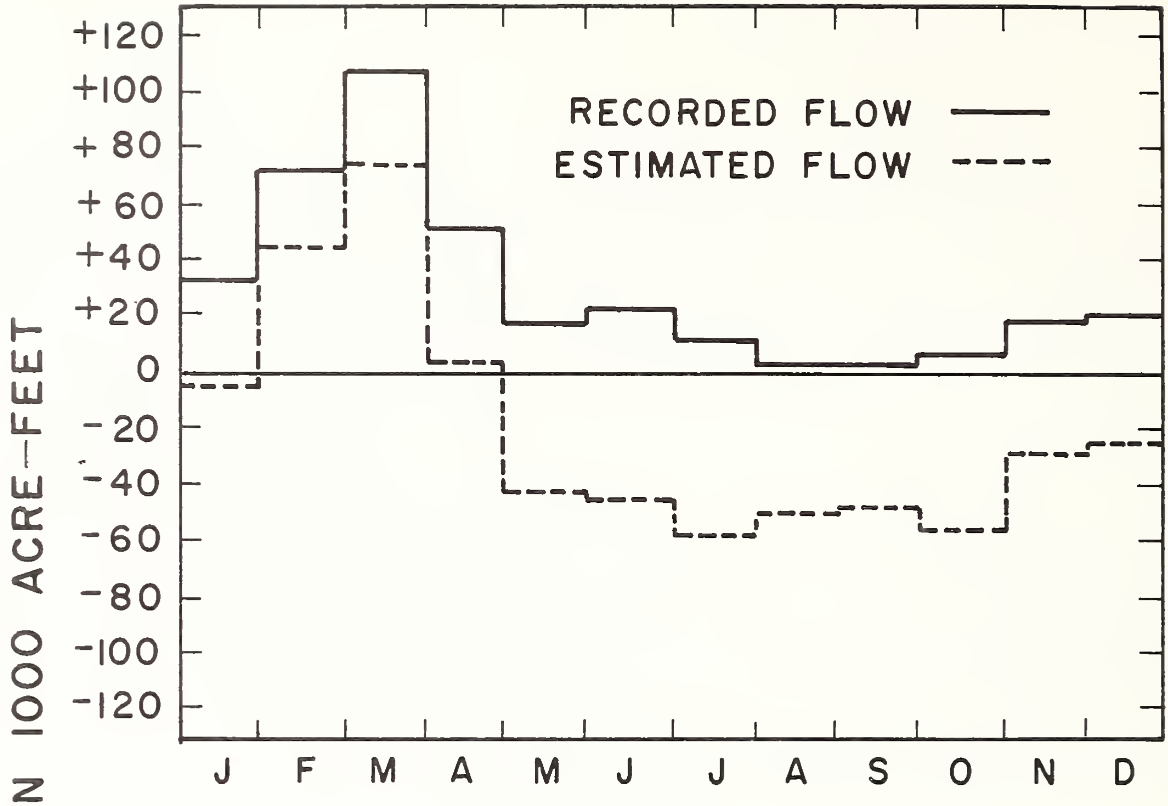

Figure. 1.

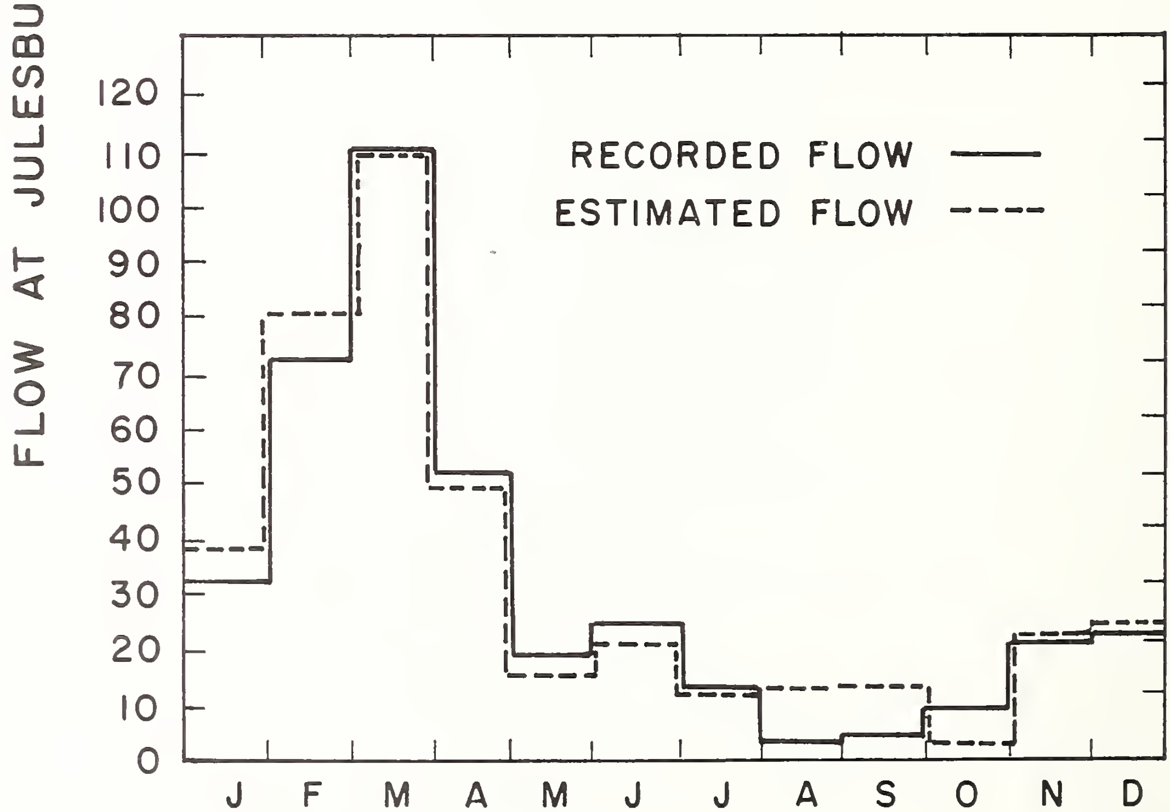

Figure 2.

Base Return Flow

Since the Maasland method of return flow calculation requires the evaluation of an infinite series, a residual always remains. When this residual is reduced to a few percentages, it can be multiplied by the average applied water over a long period of record to yield the base return flow. This insures accurate, volumetric accounting.

Channel Storage

Storage within the stream channel can be estimated by computing an average flow for the month, and using a representative cross section of the channel to establish a flow-depth-volume relationship. Hence, the change in channel storage can be estimated directly from the monthly changes in flow.

RESULTS

Figure 1 illustrates the correlation obtained if a similar study were performed on the South Platte using only the surface components in the budgetary equation. These components do not adequately describe the utilization and migration of water.

Figure 2 is the result obtained from the method described for the South Platte for the same year. That return flow of ground water must be significant is evident by the improved replication. Further, the degree of replication indicates that the proposed methods of accounting for return flows, consumptive use, and channel storage changes are efficient in the solution of the problem of the river-aquifer system.

Table 1 lists several of the major components in the budgetary equation. The calculations involve small differences between large numbers, and therefore small input errors can result in large errors of estimation.

TABLE 1.—Major components in the budgetary equation, 1948 (All values in acre-feet)

Month	Flow at Kersey	Consumptive use	Applied water	Return flow	Change in surface storage	Estimated Julesburg flow	Actual flow
Jan.	38,620	0	7,917	26,936	—325	38,547	33,600
Feb.	64,940	8,250	—1,583	23,872	4,548	81,788	72,040
Mar.	98,120	20,750	—2,000	21,203	4,314	107,991	108,600
Apr.	94,750	39,000	28,250	23,342	—3,737	47,799	51,130
May	101,800	62,500	120,083	37,868	—1,736	14,821	17,730
Jun.	96,250	78,750	107,184	47,267	16	20,363	23,510
Jul.	9,090	97,750	50,822	45,089	—7,864	11,462	11,900
Aug.	8,390	87,000	63,601	44,980	—1,141	11,764	2,460
Sep.	7,790	56,500	65,581	42,437	—367	11,817	2,440
Oct.	16,990	34,000	22,009	39,959	1,601	—618	7,750
Nov.	24,820	15,500	2,798	34,341	1,805	20,903	19,410
Dec.	27,070	0	8,750	31,172	283	23,035	20,460

CONCLUSIONS

A comparison of figure 1 with figure 2 provides substantial proof that the ground water and surface water are one contiguous supply. Surface water correlations alone do not yield meaningful estimates of the surface outflow at the downstream gage. Inclusion of the ground water return flows in the budgetary equation significantly improves the replication of the observed outflows. Use of the Maasland method produces satisfactory results in computing return flows.

Further analysis including more of the minor water losses in detail and utilizing statistical techniques of fitting might well increase the accuracy of the model. The relative advantages of using a consumptive use pattern versus a pumping pattern should also be studied.

ACKNOWLEDGMENT

The author wishes to particularly thank Robert E. Glover who gave of his time, donated his camera for the preparation of slides used in the visual presentation, and who contributed substantially to the ideas and developments expressed in this paper.

SUBSURFACE IMPLICATIONS FROM SURFACE HYDROGRAPH ANALYSIS[1]

Willard M. Snyder[2]

INTRODUCTION

The hydrographs of surface streamflow are usually treated analytically after arbitrary separation into two or more categories of flow. Normally streamflow supposedly resulting from surface runoff is separated from streamflow supposedly resulting from ground-water drainage. Occasionally a third category of subsurface flow is added. On small watersheds sometimes the base, or ground-water flow, is absent. Considering the normal case, the separation into two categories, surface and base flow, is arbitrary and assumptive. Consequently, any conclusions based on results of subsequent analysis of the separated portions are colored, to say the least, by the initial assumptions. Additionally, valuable information may be ignored.

Computer based techniques can be used today to avoid initial assumptions in hydrograph analysis. New and more informative techniques of separation are possible. From analytically separated hydrographs, it should be possible to gain a great deal of insight into the processes by which a watershed receives, stores, conveys, and releases water to form stream discharge. It should be possible to interpret these processes in terms of physical characteristics of the watersheds.

In the following presentation several departures from conventional procedure in hydrograph analysis will be evident.[3] Among these are the following:

1. The total response of the streamflow to storm input is analyzed. Only flow antecedent to the storm under analysis is separated.

2. The hydrograph is open ended. By this is meant that it is not necessary to specify a time at which flow of a certain category becomes zero.

3. The surface hydrograph is separated into a large number of time-of-flow categories after, and based upon, the storm analysis.

4. Every storm in a watershed record can be treated. It is not necessary to limit so-called unit hydrograph derivation to a few selected storms.

The procedures to be described in this paper represent a complete "package" of computer programs for processing of hydrologic records. The techniques of hydrograph analysis will be treated in some detail. However, it will be necessary to describe briefly each step in the data processing procedures so that the hydrograph analysis can be visualized in proper context.

STORM DECISION

The computer package of techniques is intended for systematic and uniform application to hydrologic records in units of one month. Human and subjective definitions in data processing are to be avoided. Consequently, the first requirement is for computer decision that a "storm" has in fact occurred. A simple decision algorithm has been developed which allows invariant definition of storm rainfall amount and storm rainfall duration.

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[3] Linsley, R. K., Jr., Kohler, M. A., and Paulhus, J. L. H. Applied Hydrology. 689 pp. 1949. McGraw-Hill, N. Y.

The storm decision algorithm is shown schematically in figure 1. The essential elements are two boundary lines on a chart of accumulated rainfall versus time. Starting at any point in time in the record which is the beginning of a rainfall event, the computer examines accumulating amounts. If the amount falls below the lower boundary,

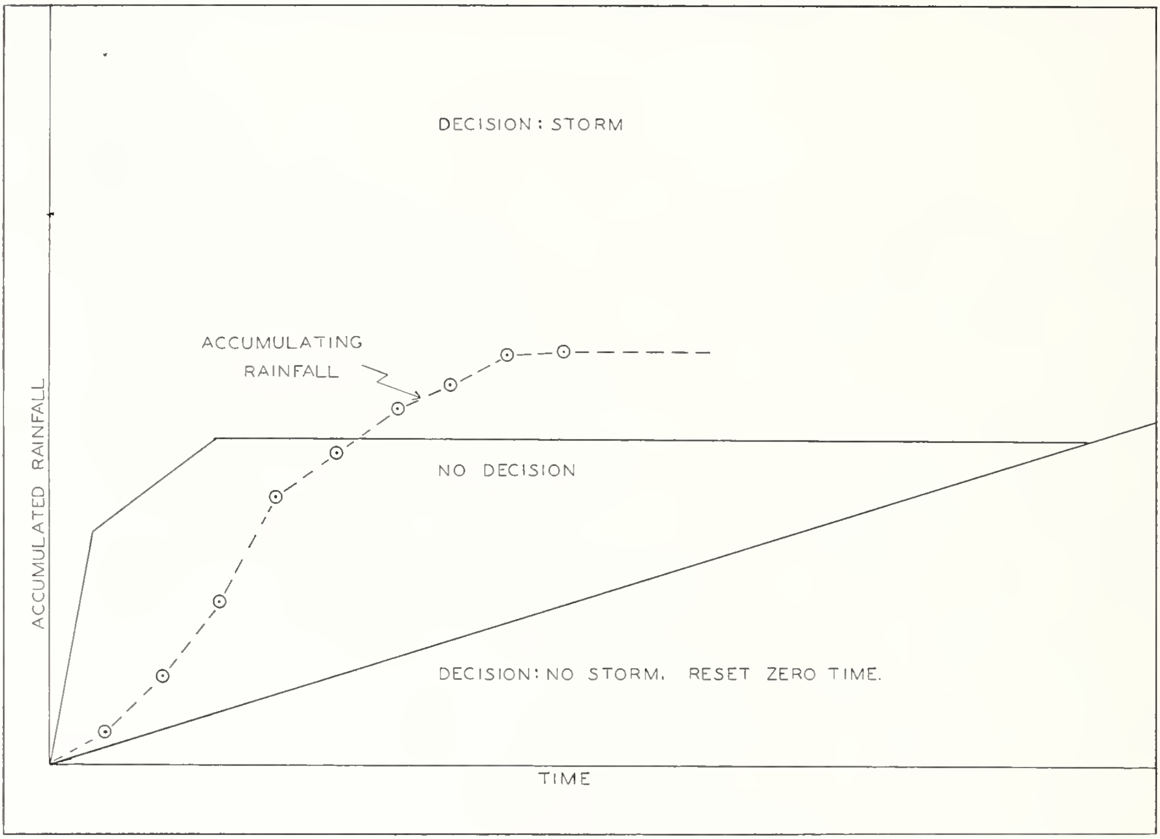

Figure 1.—Storm decision schematic.

a decision is made that the event does not constitute a storm and the time initial-point advances in the record. If the accumulating rainfall falls above the lower boundary, but below the upper boundary, it is in a "no decision" zone on the chart. This is a closed zone and eventually, as time advances, a decision of "storm" or "no storm" must be made. If the accumulating rainfall crosses the upper boundary a storm is declared.

The end of the storm is marked when the accumulating line again crosses the lower boundary. The actual end of the storm may be at this time, or at the preceding last rainfall increment. Storm amount and duration are thus simultaneously and systematically determined. Both the upper and lower boundaries can be set at will, though it seems reasonable to set the lower boundary near an estimated saturation infiltration rate for the drainage area.

STORM VOLUME OF RUNOFF

Following the detection of a storm rainfall amount the beginning time is referred by the computer to the streamflow record. The current storm is first isolated as is shown in figure 2. The process of isolating consists in removing the antecedent streamflow projected forward in time beneath the current storm. Figure 2 shows that the residual hydrograph is the total response to the storm event and is open ended. The volume is determined by a bounded figure, except at the tail of the recession when an ending time must be chosen. This time is not critical, but the open-end ordinate should be small.

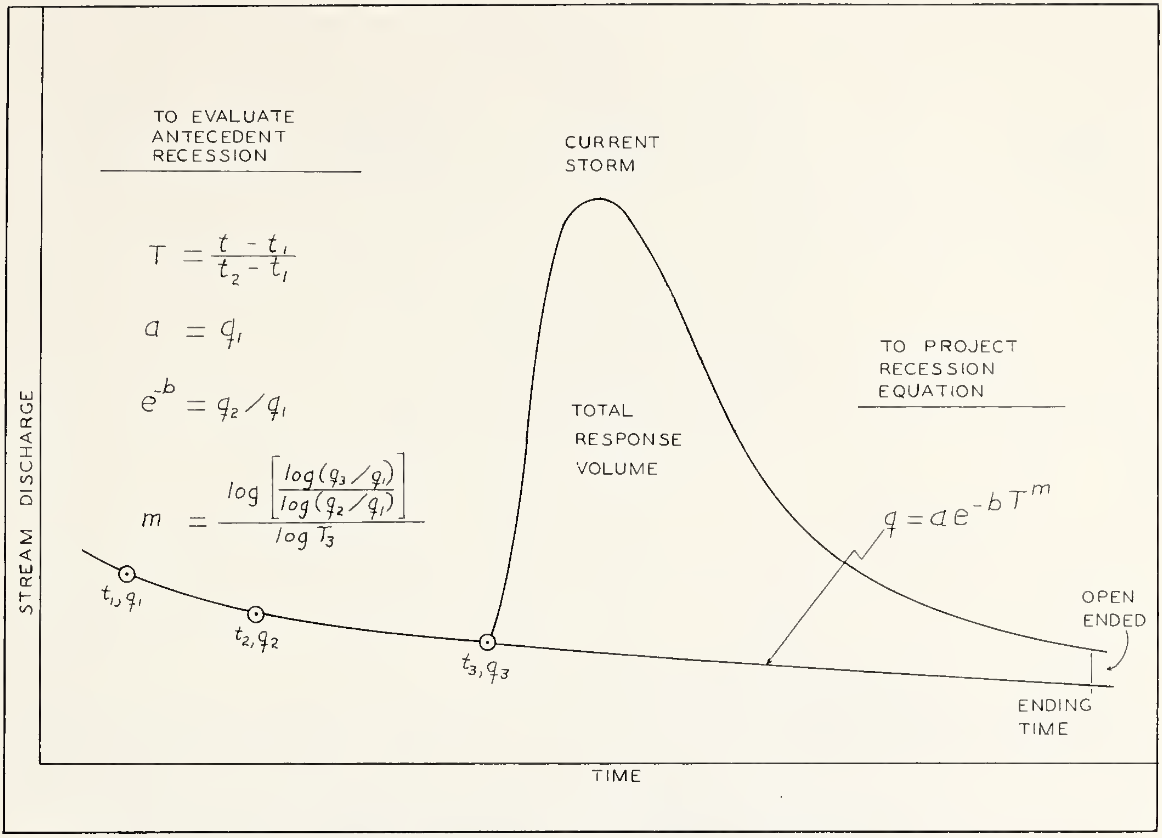

Figure 2.—Isolation of current storm response.

RAINFALL EXCESS DETERMINATION

The volume of total hydrograph response must be balanced with an equal volume of rainfall in excess of some loss rate. The excess is computed as shown in figure 3. Loss is based on a sequential finite difference equation as shown. The loss varies between an upper and a lower limit that are considered known properties of the drainage area. Approach to these limits is asymptotic with the change in loss in time increment Δt also being propotional to rainfall in Δt. An initial value of loss at the beginning of the storm is assumed, and the loss throughout the storm is computed. Rain in excess of the loss is totaled and compared with the volume of streamflow. The initial loss value is varied systematically upward or downward until the rainfall excess equals the volume of streamflow.

The main intent of this loss curve was to improve the traditional method of considering loss, or infiltration, constant during a storm. At the same time, a simple equation had to be developed that required only direct sequential computation regarding the rainfall increment. The main purpose of the loss equation was to produce an estimate of the rainfall excess by increments throughout the duration of the storm.

LEAST SQUARES DERIVATION OF TRANSFER FUNCTION

The steps in computer processing of data to this point have identified a rainfall occurrence as a storm, isolated and determined the volume of total streamflow response, and estimated the distribution of rainfall excess. The next step is to determine that process of distribution in time by which the excess becomes streamflow. In older hydrologic terminology this distributive function was called a unit hydrograph. Since total stream response is under analysis in systems terminology, this function will be called a transfer function. The principle of linearity of superposition will still be followed.

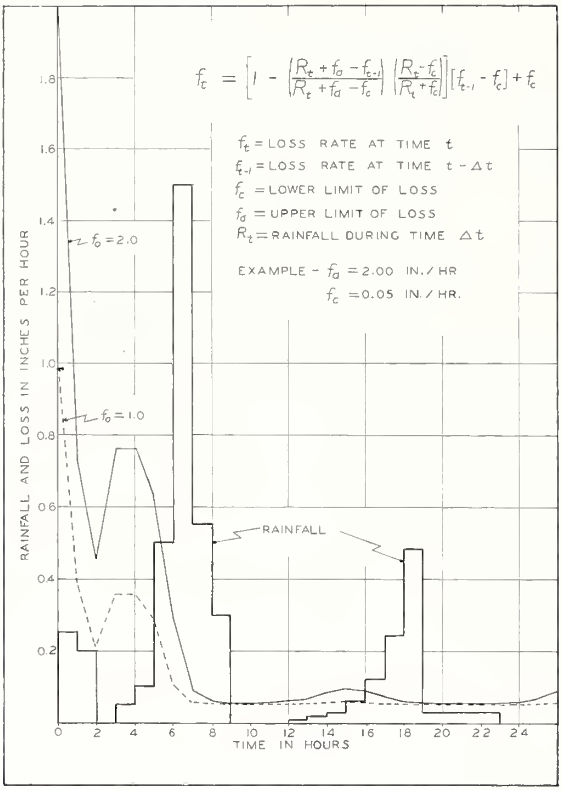

Figure 3.—Determining excess rainfall.

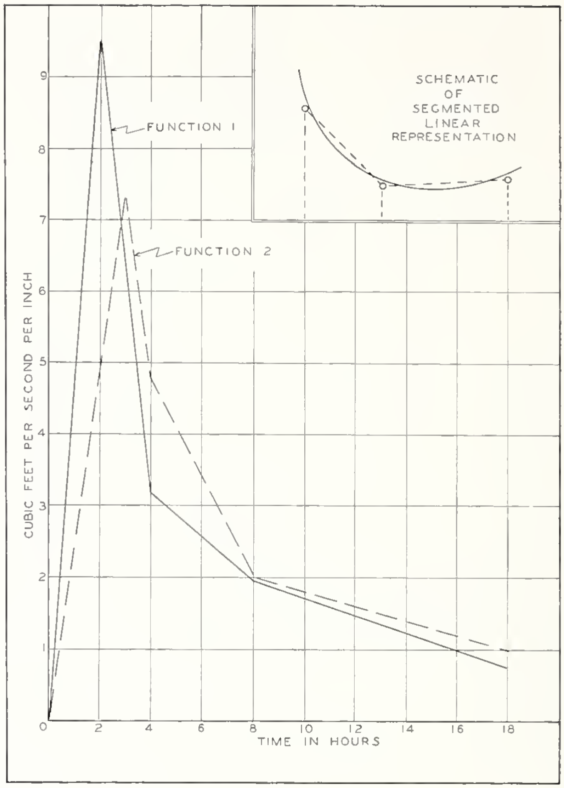

Figure 4.—Typical transfer functions.

Selection of Unit of Time

To arrange rainfall excess and storm hydrograph response data in form suitable for analytical determination of the transfer function, a unit of time must be selected for the data. The unit should be short, say 1 hour for moderate-sized watersheds down to 5 minutes for small watersheds. A short unit can be used for intensive summer storms and a longer unit for prolonged cyclonic storms.

The unit of time selected controls the following elements in data organization.

1. Increments of rainfall excess are determined by successive units of the time.

2. Ordinates of the storm hydrograph are selected at this time interval.

3. The total base of the storm hydrograph is an integral number of time units. (This feeds back to determination of hydrograph volume.)

4. Ordinates of the transfer function are found at this interval of time.

5. The base of the transfer function is the same as the base of the storm hydrograph.

Convolution

The process of distributing sequential elements in time is expressible by the well-known convolution integral. This process of integration requires that both the input function, the rainfall excess, and the transfer function be mathematically continuous in time. It is difficult to express rainfall as a time-continuous function, and the mathematical form of the transfer function is usually not known. Consequently, discrete forms of the convolution process are normally used. This process is shown in the set of equations 1.

In this set of equations, N simultaneous equations express N ordinates of the total stream response hydrograph. These N ordinates are known following the isolation of

Equations 1

Units of Time	Ordinates of the Transfer Function	Discrete Convolution Time Increments of Rainfall Excess r_1 r_2 r_3 r_4		Total Stream Response
1	u_1	k_1u_1	=	Q_1
2	u_2	$r_1u_2 + r_2u_1$	=	Q_2
3	u_3	$r_1u_3 + r_2u_2 + r_3u_1$	=	Q_3
4	u_4	$r_1u_4 + r_2u_3 + r_3u_2 + r_4u_1$	=	Q_4
5	u_5	$r_1u_5 + r_2u_4 + r_3u_3 + r_4u_8$	=	Q_5
6	u_6	$r_1u_6 + r_2u_5 + r_3u_4 + r_4u_3$	=	Q_6
7	u_7	$r_1u_7 + r_2u_6 + r_3u_5 + r_4u_4$	=	Q_7
—	—			
—	—			
—	—			
N-3	u_{N-3}			
N-2	u_{N-2}			
N-1	u_{N-1}			
N	u_N	$r_1u_N + r_2u_{N-1} + r_3u_{N-2} + r_4u_{N-3}$	=	Q_N

the current storm from the antecedent recession. The increments of rainfall excess are known following trial and error application of the loss equation. Therefore, only the set of ordinates, u_t, of the transfer function are unknown.

Since the number of unknowns and the number of equations are equal, the set of equations could be solved for the unknown ordinates. In fact, a triangular condition exists at the beginning of the storm so that such a solution would be extremely easy. However, a different method of solution is required. Many errors and indeterminant elements are present. Therefore, a method of solution should be used which produces some average or optimum set of u_t's in the presence of such errors. Solution by the method of least squares can accomplish such an "averaging" solution.

For an averaging process to work, the number of unknowns must be reduced to some order less than the number of equations. This reduction is possible by interpolating for some of the u_t's from adjacent values. Many types of interpolation are possible, from simple linear to polynomial or trigonometric forms. The linear forms lead to segmented functions, are easy to use with nonuniform spacing of ordinates, and are the forms presented for illustration here.

Form of the Transfer Function

Two typical transfer functions are shown in figure 4. These represent stream response to a single unit of input rainfall excess occurring during the first unit of time, here taken to be 1 hour. These segmented forms are somewhat idealized. Actual transfer function could, for example, have multiple peaks. Minor irregularities causing a departure from a smooth recession may also occur. The mathematical form of this transfer function is usually unknown and mathematically continuous forms are usually limited to simple assumptive forms such as gamma function, or else are represented by infinite series, as in Fourier analysis. The simple continuous forms cannot usually be used with actual hydrographs showing natural irregularities from the idealized form.

The linear segmented transfer functions should be considered approximations for some underlying but unknown smoothly continuous function. The form of this substitution is shown schematically in the insert in figure 4. It should be noted particularly that the ordinates of the segmented function do not equal the corresponding ordinates of the underlying smooth function. Even the selected ordinates at angle points have

small deviations. The total set of linear ordinates must be considered an optimal substitution for the ordinates of the smooth function.

To define such a set of straight lines, the location of the angle points must be specified. Practically, this is not of great concern. As long as angle points are closely spaced across the zone of the maximum function, the placement is not critical. Also, the small curvature of the tail of the recession allows wide spacing of angle points.

Several advantages derive from using such a set of straight-line segments. Any function can be so represented to some degree of precision, including extreme departures from the idealized form. It is not necessary to assume any functional form. Instead, solution for function values at discrete points will be shown later. In fact, the use of straight-line segments reduces the discrete convolution to a reduced linear system capable of easy solution by least squares. Perhaps the most important concept is that one can derive numeric values of some mathematical continuity without stating an explicit mathematical function to represent that continuity.

Analytical Transformation of the Transfer Function

The approximating form of the transfer function, shown by the straight-line segments of figure 4, will now be combined with the discrete convolution equations 1. First, however, the algebraic form of the substitution in figure 4 will be given. This is ex-

Equations 2

Units of Time	Ordinates of the Transfer Function		Approximating Linear Segments	Angle Points
1	u_1	=	$1/2\ 0_2 + E_1$	
2	u_2	=	$0_2 + E_2$	*
3	u_3	=	$1/2\ (0_2 + 0_4) + E_3$	
4	u_4	=	$0_4 + E_4$	*
5	u_5	=	$0_5 + E_5$	*
6	u_6	=	$0_6 + E_6$	*
7	u_7	=	$1/2\ (0_6 + 0_8) + E_7$	
8	u_8	=	$0_8 + E_8$	*
9	u_9	=	$1/3\ (20_8 + 0_{11}) + E_9$	
10	u_{10}	=	$1/3\ (0_8 + 20_{11}) + E_{10}$	
11	u_{11}	=	$0_{11} + E_{11}$	*
12	u_{12}	=	$1/4\ (30_{11} + 0_{15}) + E_{12}$	
13	u_{13}	=	$1/4\ (20_{11} + 20_{15}) + E_{13}$	
14	u_{14}	=	$1/4\ (0_{11} + 30_{15}) + E_{14}$	
15	u_{15}	=	$0_{15} + E_{15}$	
16	u_{16}	=	$1/5\ (40_{15} + 0_{20}) + E_{16}$	
17	u_{17}	=	$1/5\ (30_{15} + 20_{20}) + E_{17}$	
18	u_{18}	=	$1/5\ (20_{15} + 30_{20}) + E_{18}$	
19	u_{19}	=	$1/5\ (0_{15} + 40_{20}) + E_{19}$	
20	u_{20}	=	$0_{20} + E_{20}$	*

pressed by the set of equations 2. The transformation is accomplished by specifying ordinates at the angle points where linear segments connect. At these points the ordinates plus some unknown error equal the ordinates of the original transfer function. The intermediate ordinates of the linear segments can be inserted directly by simple linear interpolation. These interpolated values plus some unknown error also equal the corresponding ordinates of the original transfer function.

The enforcement of an averaging process is evident in equations 2. The 20 unknown discrete ordinates of the original transfer function have been reduced to 8 unknown ordinates of the approximating linear segments. The ordinate 0_{11}, for example, now occurs in six of the equations, and a method of solution, such as by least squares, must

produce some best average value in these six. Specifically, the substitution of linear segments is not made by drawing chords across selected areas of the original function. All ordinates of the approximating function, including those at angle points, will differ from the original function. In fact, the placement of the approximating function over the original function can only be accomplished by specifying some optimization process operating on the ordinate errors, E.

If a transfer function $u_t (t = 1, 20)$ were known, the set of equations 2 could be solved directly by least squares to get an optimum set of linear ordinates, the 0's. However, in practical storm analysis, the u_t's are not known, and the set of ordinates, 0, must be derived directly from the storm rainfall and streamflow.

Solution for the Linear-Segmented Hydrograph

The transforming equations 2 may be substituted into the discrete convolution equation 1. The result is a system of linear equations relating rainfall excess, the transformed transfer function, and the total stream response hydrograph. This set is shown as equations 3.

Equations 3

Units of Time	r_1	r_2	r_3	r_4	Total Stream Response
1	$\frac{1}{2} r_1 0_2$				$= Q_1$
2	$r_1 0_2$	$+ \frac{1}{2} r_2 0_2$			$= Q_2$
3	$\frac{1}{2} r_1 (0_2 + 0_4)$	$+ r_2 0_2$	$+ r_3 0_2$		$= Q_3$
4	$r_1 0_4$	$+ \frac{1}{2} r_2 (0_2 + 0_4)$	$+ r_3 0_2$	$+ \frac{1}{2} r_4 0_2$	$= Q_4$
5	$r_1 0_5$	$+ r_2 0_4$	$+ \frac{1}{2} r_3 (0_2 + 0_4)$	$+ r_4 0_2$	$= Q_5$
6	$r_1 0_6$	$+ r_2 0_5$	$+ r_3 0_4$	$+ \frac{1}{2} r_4 (0_2 + 0_4)$	$= Q_6$
7	$\frac{1}{2} r_1 (0_6 + 0_8)$	$+ r_2 0_6$	$+ r_3 0_5$	$+ r_4 0_4$	$= Q_7$
8	$r_1 0_8$	$+ \frac{1}{2} r_2 (0_6 + 0_8)$	$+ r_3 0_6$	$+ r_4 0_5$	$= Q_8$
9	$\frac{1}{3} r_1 (20_8 + 0_{11})$	$+ r_2 0_8$	$+ \frac{1}{2} r_3 (0_6 + 0_8)$	$+ r_4 0_6$	$= Q_9$
10	$\frac{1}{3} r_1 (0_8 + 20_{11})$	$+ \frac{1}{3} r_2 (20_8 + 0_{11})$	$+ r_3 0_8$	$+ \frac{1}{2} r_4 (0_6 + 0_8)$	$= Q_{10}$
11	$r_1 0_{11}$	$+ \frac{1}{3} r_2 (0_8 + 20_{11})$	$+ \frac{1}{3} r_3 (20_8 + 0_{11})$	$+ r_4 0_8$	$= Q_{11}$

In equations 3 the ordinate errors shown in equations 2 have not been shown explicitly. To produce an "averaged" solution of the equations 3, some residual error must also be considered attached to each one of the set. The transform errors of equations 2 are thus combined with the discrete convolution errors of equations 3.

The left side of equations 3 can be rearranged. An arrangement by collection of terms with common ordinates is shown in table 1. The common ordinates are shown factored out and placed at the head of the respective columns. The equal and the plus signs between major segments of the equations are also omitted.

Table 1 provides an arrangement of data, the rainfall-excess terms and stream-hydrograph ordinates, that allows direct application of the method of least squares for solution of the set of transfer function ordinates, 0. The values of eight ordinates will be averaged over 20 stream ordinates by least squares definition. It can be seen that the system of interpolation for the linear-segmented transfer function has produced a set of numeric operators on the rainfall-excess terms. Such operations on the excess terms produce the X's of a multiple-regression equation. The storm-hydrograph ordinates, Q, are obviously the Y's of a multiple regression.

The example shown here is for a matrix dimension produced by 20 storm ordinates and four periods of rainfall excess. However, the basic concepts are general. Any number of storm ordinates could be used with any number of rainfall terms. Other transforms of the transfer function than that shown in equations 2 could also be used. The entire process can be readily programed for application to all storms of record by electronic

TABLE 1.--Least Squares Solution for Transfer Function

Time	O_2	O_4	O_5	O_6	O_8	O_{11}	O_{15}	O_{20}	Total Stream Response
	Transfer Function Ordinates								
1	$\frac{1}{2}r_1$								Q_1
2	$r_1 + \frac{1}{2}r_2$								Q_2
3	$\frac{1}{2}r_1 + r_2 + \frac{1}{2}r_3$	$\frac{1}{2}r_1$							Q_3
4	$\frac{1}{2}r_2 + r_3 + \frac{1}{2}r_4$	$r_1 + \frac{1}{2}r_2$							Q_4
5	$\frac{1}{2}r_3 + r_4$	$r_2 + \frac{1}{2}r_3$	r_1						Q_5
6	$\frac{1}{2}r_4$	$r_3 + \frac{1}{2}r_4$	r_2	r_1					Q_6
7		r_4	r_3	$\frac{1}{2}r_1 + r_2$	$\frac{1}{2}r_1$				Q_7
8			r_4	$\frac{1}{2}r_2 + r_3$	$r_1 + \frac{1}{2}r_2$				Q_8
9				$\frac{1}{2}r_3 + r_4$	$\frac{2}{3}r_1 + r_2 + \frac{1}{2}r_3$	$\frac{1}{3}r_1$			Q_9
10				$\frac{1}{2}r_4$	$\frac{1}{3}r_1 + \frac{2}{3}r_2 + r_3 + \frac{1}{2}r_4$	$\frac{2}{3}r_1 + \frac{1}{3}r_2$			Q_{10}
11					$\frac{1}{3}r_2 + \frac{2}{3}r_3 + r_4$	$r_1 + \frac{2}{3}r_2 + \frac{1}{3}r_3$			Q_{11}
12					$\frac{1}{3}r_3 + \frac{2}{3}r_4$	$\frac{3}{4}r_1 + r_2 + \frac{2}{3}r_3 + \frac{1}{3}r_4$	$\frac{1}{4}r_1$		Q_{12}
13					$\frac{1}{3}r_4$	$\frac{2}{4}r_1 + \frac{3}{4}r_2 + r_3 + \frac{2}{3}r_4$	$\frac{2}{4}r_1 + \frac{1}{4}r_2$		Q_{13}
14						$\frac{1}{4}r_1 + \frac{2}{4}r_2 + \frac{3}{4}r_3 + r_4$	$\frac{3}{4}r_1 + \frac{2}{4}r_2 + \frac{1}{4}r_3$		Q_{14}
15						$\frac{1}{4}r_2 + \frac{2}{4}r_3 + \frac{3}{4}r_4$	$r_1 + \frac{3}{4}r_2 + \frac{2}{4}r_3 + \frac{1}{4}r_4$		Q_{15}
16						$\frac{1}{4}r_3 + \frac{2}{4}r_4$	$\frac{4}{5}r_1 + r_2 + \frac{3}{4}r_3 + \frac{2}{4}r_4$	$\frac{1}{5}r_1$	Q_{16}
17						$\frac{1}{4}r_4$	$\frac{3}{5}r_1 + \frac{4}{5}r_2 + r_3 + \frac{1}{4}r_4$	$\frac{2}{5}r_1 + \frac{1}{5}r_2$	Q_{17}
18							$\frac{2}{5}r_1 + \frac{3}{5}r_2 + \frac{4}{5}r_3 + r_4$	$\frac{3}{5}r_1 + \frac{2}{5}r_2 + \frac{1}{5}r_3$	Q_{18}
19							$\frac{1}{5}r_1 + \frac{2}{5}r_2 + \frac{3}{5}r_3 + \frac{4}{5}r_4$	$\frac{4}{5}r_1 + \frac{3}{5}r_2 + \frac{2}{5}r_3 + \frac{1}{5}r_4$	Q_{19}
20							$\frac{1}{5}r_2 + \frac{2}{5}r_3 + \frac{3}{5}r_4$	$r_1 + \frac{4}{5}r_2 + \frac{3}{5}r_3 + \frac{2}{5}r_4$	Q_{20}

data processing. Following such application for solution of the "angle-point" ordinates specified in table 1, the ordinates between angle points can be found by the linear interpolation indicated under "Approximating linear segments" in equations 2.

The concept of a segmented transfer function produces a linearized and discrete convolution form. This simplified form does not require much time or effort, either in preliminary data arrangement or in storm analysis, when considering systematic processing of all storms in a hydrologic record.

TIME-SEPARATED HYDROGRAPHS

The Reconstructed Hydrograph

Stream response to a storm can be computed from the increments of rainfall excess and the ordinates of a transfer function, as shown by the discrete convolution in equations 1. In fact, in earlier analysis techniques the transfer function was usually determined from only a small number of selected storms meeting idealized criteria. Its suitability for use with long-duration storms of complex rainfall pattern was determined by trial computation. The computed ordinates were compared, usually graphically, with the observed storm ordinates.

After a linear segmented transfer function has been determined by least squares, it can also be applied to the increments of rainfall excess, and the total stream response can be calculated. This process is identical to the computation of "predicted values" for comparison with "observed values" in ordinary multiple-regression analysis. The "predicted values" form a reconstructed version of the stream response. The total response above the antecedent base flow is reconstructed.

Partially Reconstructed Hydrographs

Even though partial values of the reconstructed storm hydrograph can be used to form a time-separated hydrograph, this procedure is not used. It will be developed by considering the full set of transfer function ordinates, 0, which are approximations for the u's in equations 1.

The area enclosed by the polygon defined by the ordinates 0 represents unit volume of total runoff. Even though continuity of mass was not specifically stated, it is contained in equations 1. If the time-base of the transfer function is N time units long, then all the ordinates from time zero to time N define the total volumetric distribution. However, if one considered dropping the ordinate 0_N then the partial transfer function, defined by ordinates from time zero to time N—1 must represent a reduced volume. The volume unaccounted for is the volume of water with travel time through the watershed ranging from N—1 to N. The same logic applies to any ordinate at some time T between zero and N. The partial transfer function composed of ordinates from zero to T represents the volume of water with travel time less than T; the partial transfer function from T to N represents the volume of water with flow time greater than T.

One can now consider the effect of systematic elimination of the terms in equation 1. If one drops the terms on the left sides of equations 1, which contain 0_1 (substituted for u_1), then the first four stream response ordinates are reduced. This reduction represents water with travel time from zero to one time unit. If now the 0_2 terms are additionally dropped, this further reduction represents volume of water with travel time from one to two time units. Systematic dropping of the transfer function terms produces sets of partial response ordinates. By plotting all such sets of partial ordinates under the storm hydrograph, a time-separated hydrograph is produced.

Examples of Time-Separated Hydrographs

Figure 5 shows a time-separated hydrograph constructed from the transfer function number 1 of figure 4 and the increments of rainfall excess as shown. Figure 6 shows a time-separated hydrograph constructed from the same rainfall excess but using transfer function number 2 from figure 4.

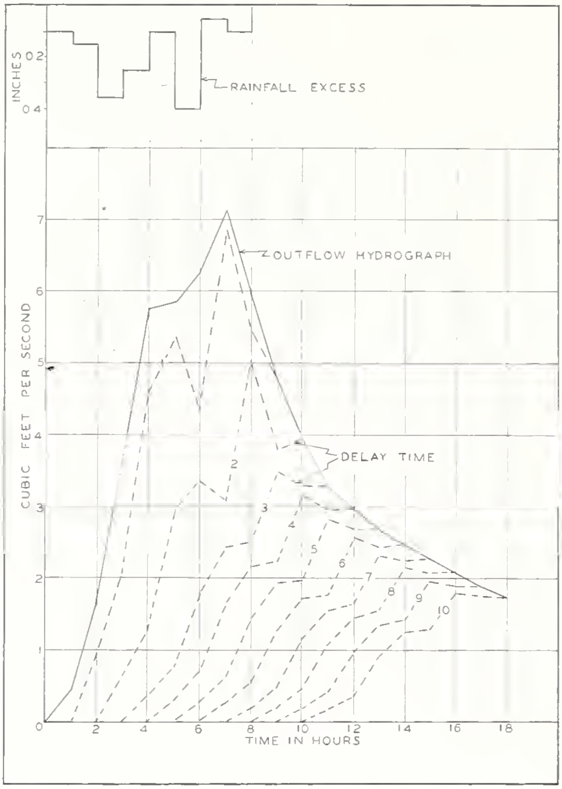

Figure 5.—Function 1 outflow hydrograph.

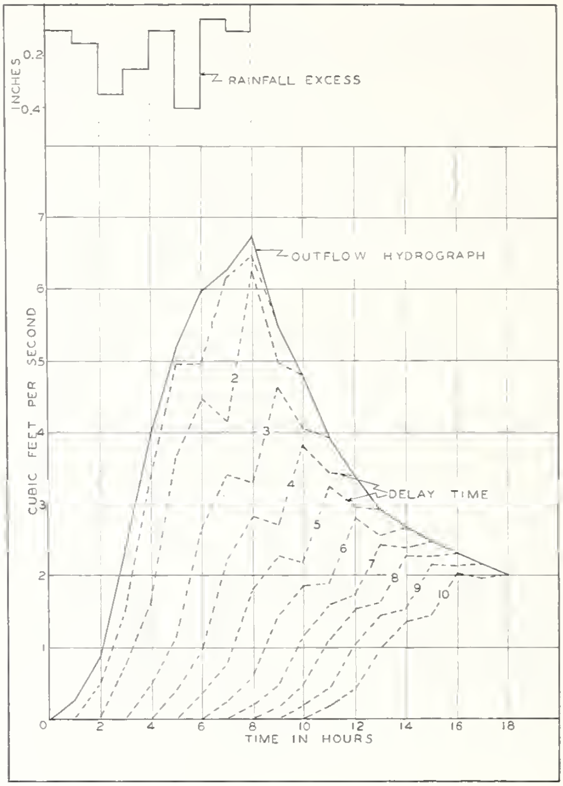

Figure 6.—Function 2 outflow hydrograph.

In figures 5 and 6, the solid bounding line represents total stream response to the input rainfall excess. As such, it represents increase in streamflow above antecedent flow regardless of whether this increase is called surface, subsurface, or ground-water runoff. The time separation lines were constructed for delay time increments of 1 hour. Areas on the graph between indicated delay times represent volumes of flow between these delay times.

Figure 5 shows a total peak slightly higher than that of figure 6 because of the higher peak ordinate of the associated transfer function. A much more striking difference, however, is found in the volumes of water with short delay times. Figure 5 contains a much greater volume in 0-to-3-hour-delay time than does figure 6. Beyond about 3-hours-delay time, the pattern of separation is similar in the two figures, though delayed flows are somewhat greater in figure 6.

The major point to be brought out in comparison of figures 5 and 6 is the separation of varying rapid response from more stable delayed response. The separation results from a purely analytical and objective procedure.

SUBSURFACE IMPLICATIONS

In summary, to this point of development the attempt has been made to portray a simple and efficient computer-oriented procedure for analysis of surface hydrographs. This procedure has the following significant features:

1. All storms in a hydrologic record for a drainage area are given systematic and unvarying treatment from selection through isolation, derivation of transfer function, and construction of time separation lines.

2. As a result of such systematic procedure, the transfer function, the increments of rainfall excess, and the time-separated hydrograph are now available for every storm.

The procedures outlined here are currently being programed electronically. Consequently, no large sets of objective storm analysis are available. However, all individual components of the data-processing package have been either separately tested or previously reported.[4]

It seems profitable to speculate at this point on some questions that might be raised after output data become available. These questions deal with possible interpretation of the output data to infer movement of water through the soil profile. They may indicate an area for cooperative effort between surface-water and ground-water hydrologists.

First, given storm analyses for rain falling on both wet and dry conditions, can variations in the transfer function and in the time-separated hydrographs be used to infer runoff sources and flow paths in the watershed?

Second, can the variations in delayed flow patterns from storms on wet ground to storms on dry ground be used to infer transmission properties of the soils in the watershed?

Third, can the volumes of water in the delayed flow categories be used to estimate recharge and detention characteristics of the soil profile?

Fourth, for an idealized or simply structured watershed, would a deterministic analysis of the watershed system produce flow categories similar to those produced by an objective hydrograph analysis?

ACKNOWLEDGEMENTS

This paper represents work on research sponsored by Oak Ridge National Laboratory, which is operated by Union Carbide Corporation for the U.S. Atomic Energy Commission. This work was in conjunction with Walker Branch Watershed Project of the ORNL Radiation Ecology Section, Health Physics Division.

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SUBSURFACE FLOW REGIMES OF A HYDROLOGIC WATERSHED MODEL[1]

C. A. Onstad and D. G. Jamieson[2]

INTRODUCTION

Parametric hydrology recognizes three major components — input, output, and the operator that acts on input to produce output. In subsurface flows, input is the difference between precipitation and overland flow, output is the sustained baseflow, and the operator is a complex, nonlinear function. Even if the assumption of linearity is made, two of the components must be known to calculate the third. The response of baseflow to a precipitation event has long been nebulous because the input and the operator are unknown. Furthermore, only the sustained portion of the output is measured, as the remainder is obscured by surface runoff. This has given rise to a multitude of arbitrary techniques for separating baseflow, all of which are outdated. The investigation presented here synthesizes the subsurface seepage system and justifies each flow regime by hydrograph analysis.

BACKGROUND

The usual approach to baseflow analysis is to derive a relationship between discharge and storage. This relationship has led to the idea that the baseflow system can be ap-

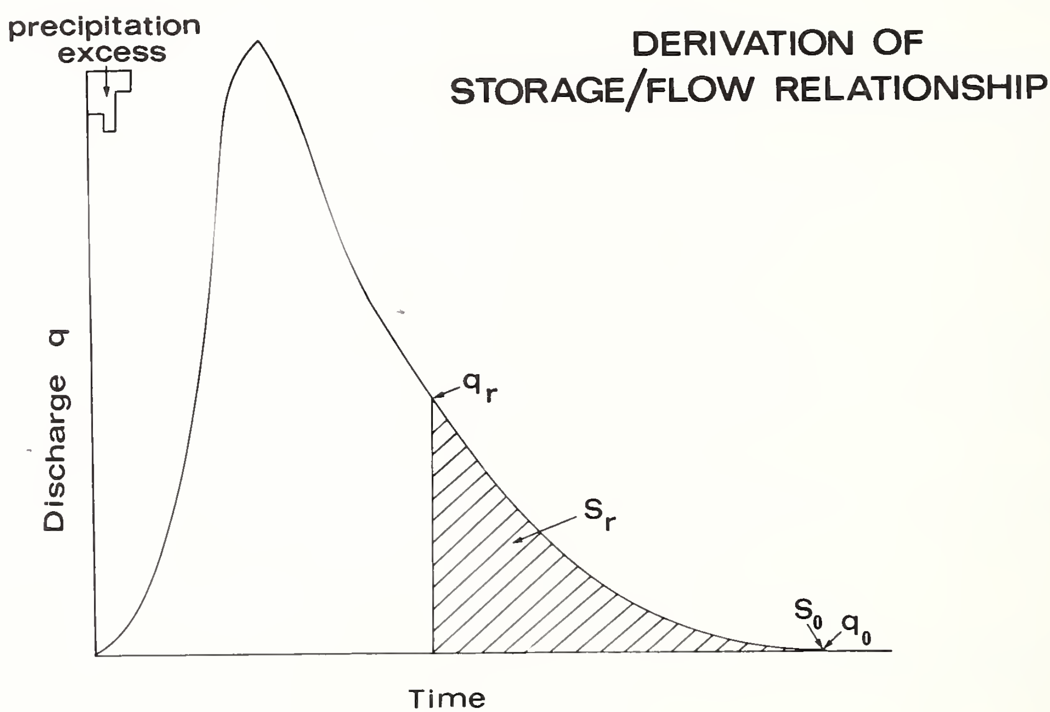


Figure 1.—Derivation of storage flow relationship.

[1] Contribution from Soil and Water Conservation Research Division, Agricultural Research Service, U. S. Department of Agriculture.

[2] Formerly Agricultural Engineer and Hydraulic Engineer, U. S. Dept. of Agr. Hydrograph Laboratory, Beltsville, Md.

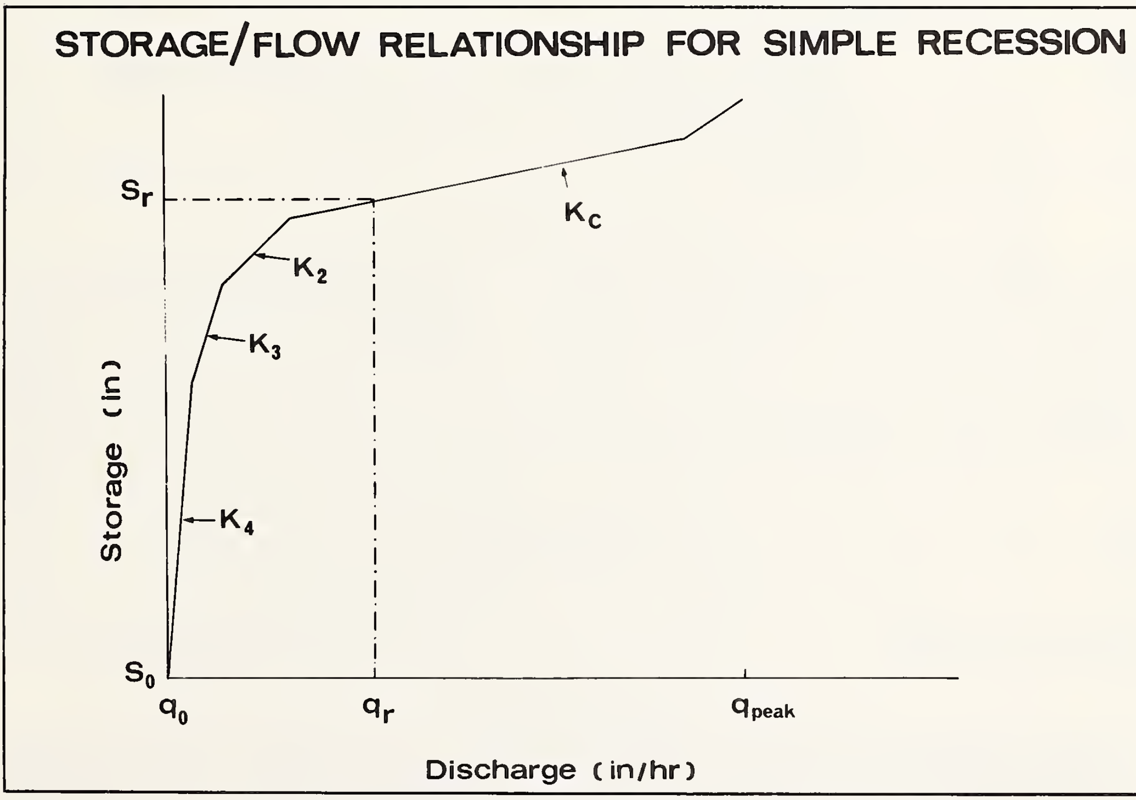
proximated by a single linear reservoir. Few researchers, however, have been able to derive the inflow to this reservoir as it cannot be assumed equal to the rate of surface infiltration; thresholds and lateral seepage in the soil's upper horizons add to the complexity of deriving the recharge.

Kadoya (*6*) has assumed the recharge rate equal to the rate of infiltration after prolonged wetting as defined by Horton (*5*). The simplification is made that recharge begins after the field capacity is exceeded in the upper 10 cm. Sugawara (*10*), who had previously conceived the idea of representing the land phase of the hydrologic cycle by cascading reservoirs, assumed the recharge to be constant throughout the period of rainfall. A similar arbitrary method of delegating a portion of infiltration to recharge is used in the Stanford Watershed Model IV (*2*).

HYDROGRAPH ANALYSIS

Throughout this investigation, data from an ARS experimental watershed at Coshocton, Ohio, were used. The simulation was made on a storm basis and later expanded to synthesize a period of continuous record.

One classical method of deriving the reservoir-storage coefficient for a particular watershed is to reverse integration of a simple hydrograph. Starting at a known or assumed point of zero flow and storage on the recession limb, q_0 and S_0, at any time, the amount of excess water, S_r, still contained by the system is plotted against the corresponding instantaneous discharge q_r (figs. 1 and 2.).



igure 2.—Storage flow relationship for simple recession.

For watershed 94 at Coshocton, the relationship between storage and flow approximated to a series of five straight lines. Subsequent analysis indicated that these relationships represent the various flow regimes of the hydrologic system's land phase. The gradients of these linear relationships are the storage coefficients (K) corresponding to the five flow regimes, (K_1, K_c, K_2, K_3, K_4). The storage coefficients K_2, K_3, and K_4 per-

tain to subsurface flows and K_1, which is evident although not determinable, corresponds to overland flow. K_c is the storage coefficient of the common channel network through which all excess water has to pass to reach the watershed exit. The three subsurface flow regimes were designated quick-return flow (K_2), delayed-return flow (K_3), and prolonged-return flow (K_4). The higher K-values indicate a more devious path of excess water between the points of entry and exit. The terminology was selected to describe their response time rather than the boundary conditions.

One or more of the flow regimes may not be evident depending on the relative size of the watershed, antecedent conditions, and the nature of the rainfall input. A simple example would be a short, intense summer storm on a dry watershed resulting in almost entirely overland flow with subsequent channel flow; whereas, a long-duration, low-intensity storm may yield only subsurface flow and channel flow (fig. 3). Superimposed

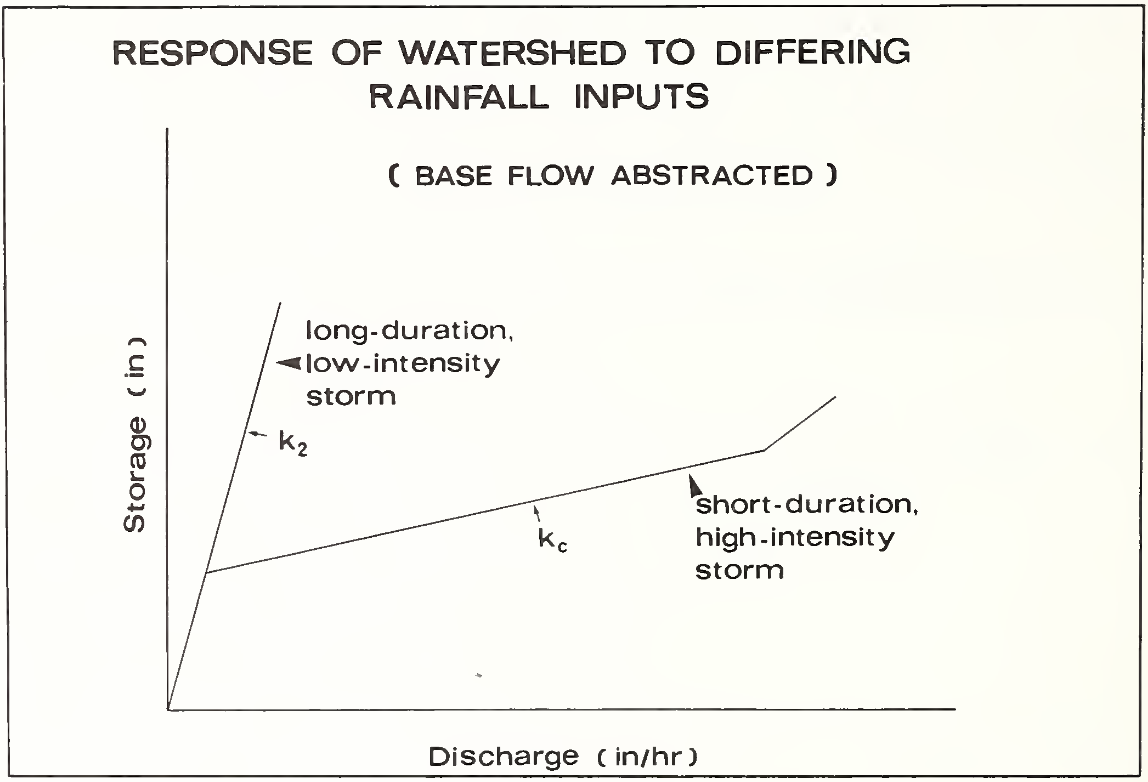


Figure 3.—Response of watershed to differing rainfall inputs.

on the flow regimes from this event is the contribution of previous events draining from the lower reservoirs.

SIMULATION

Before simulation of the observed flows can be made, a conceptual model is necessary to envisage the boundary conditions of each flow regime (fig. 4). The model portrays the precipitation input being divided into the lateral overland flow, OF, and downward infiltration, f_1. The infiltration is the input to the soil's uppermost reservoir, which is defined by the first impeding layer. Quick-return flow, QRF, is the lateral exhaustion from this reservoir. The downward seepage, f_2, through the first impeding layer forms the recharge to the next reservoir that is bounded by the impeding layer, B2, above and by the saturated zone below. Delayed-return flow, DRF, is the lateral flow out of this reservoir. The downward seepage, f_3, is the recharge to the saturated zone. Lateral flow from the saturated zone is designated prolonged-return flow.

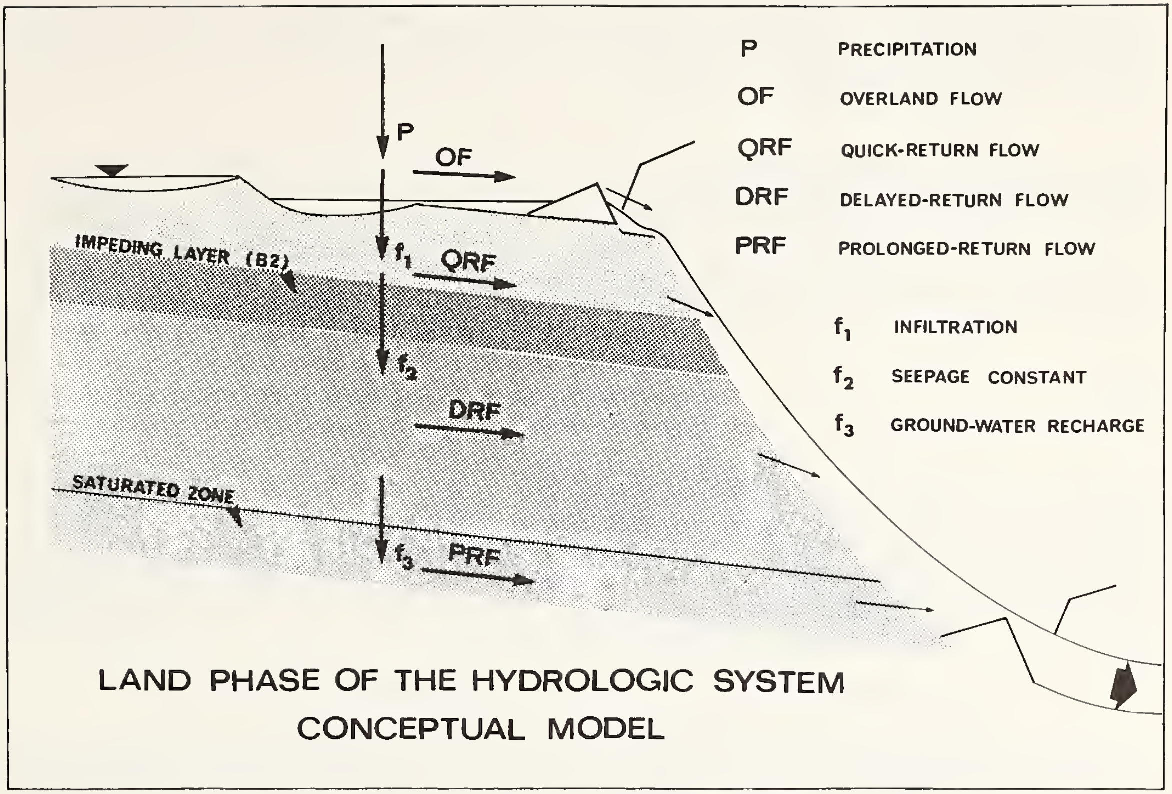


Figure 4.—Land phase—conceptual model.

This conceptual model visualizes the land phase of the hydrologic system as a combination of reservoirs in parallel and in series, the output from one forming the input to the next. Consequently, it is necessary to start with infiltration. Although any infiltration model could have been used, we used the empirical formula of Holtan (*4*) as it already makes use of the soil's upper horizon as a nonlinear reservoir. The model is nonlinear as it acknowledges that there is a feedback in the form of a complex infiltration/storage-available connection. In addition, the seepage through the impeding layer, f_2, is limited to water in excess of plant-available water capacity, C_2, which constitutes a threshold criterion. The form of Holtan's equation is:

$$f = a\, S_a^{1.4} \text{ below the threshold value,} \tag{1}$$

and $$f = a\, S_a^{1.4} + f_2 \text{ above the threshold value,} \tag{2}$$

where f = capacity rate of infiltration,
f_2 = seepage rate through impeding layer,
a = surface penetration index,
S_a = storage currently available above the restricting horizon.

Quick-return flow is assumed to be a linear exhaustion function above the same threshold criterion as the downward seepage through the impeding layer. The storage, S_2, at any time $t + 1$, is:

$$(S_2)_{t+1} = (S_2) + (f_1 - f_2) \cdot \Delta t - q_2 \cdot \Delta t, \tag{3}$$

where suffix t denotes time, Δt, the time increment, and q_2 is the lateral outflow above the restricting horizon. Therefore, the storage available, S_a, is the difference between the total porosity and S_2.

The seepage through the impeding layer, f_2, forms the input to the delayed-return reservoir. This recharge continues at a constant rate so long as the storage, S_2, exceeds

the threshold, C_2. The delayed-return flow also contains a threshold storage value which must be satisfied before lateral flow can begin. The storage status, S_3, in the delayed-return reservoir at any time t + 1 is:

$$(S_3)_{t+1} = (S_3)_t + (f_2 - f_3) \cdot \Delta t - \overline{q}_3 \cdot \Delta t, \tag{4}$$

where f_3 is the rate of recharge to the saturated zone. The rate of lateral outflow, q_3, is a linear function of the storage in excess of the threshold value, C_3. When S_3 becomes less than C_3, equation (5) is simplified to:

$$(S_3)_{t+1} = (S_3)_t + (f_2 - f_3) \cdot \Delta t. \tag{5}$$

The constant seepage rate, f_3, through the delayed-return reservoir recharges the prolonged-return reservoir so that the latter's storage, S_4, at any time t + 1 is:

$$(S_4)_{t+1} = (S_4)_t + f_3 \cdot \Delta t - \overline{q}_4 \cdot \Delta t, \tag{6}$$

where q_4 is the lateral outflow and a linear function of S_4. When the recharge, f_3 is zero,

$$(S_4)_{t+1} = (S_4)_t - \overline{q}_4 \cdot \Delta t. \tag{7}$$

In general, all lateral flows have been routed by two linear reservoirs. The first reservoir is a conceptual reservoir representing the particular surface or subsurface flow (K_1, K_2, K_3, or K_4) and the second, into which all the first-phase reservoirs cascade, is that of the channel network (K_c). Therefore, the routing procedure is essentially two unequal linear reservoirs similar to that of Singh (8); however, in practice the influence of channel storage on the baseflow regimes is so slight (K_3 and $K_4 >> K_c$), that these lower regimes were approximated by a single linear reservoir.

APPLICATION ON COSHOCTON WATERSHED 94 (2.37 sq. mi.)

Most models treat the watershed as a lumped system, whereas the prototype is a distributed system. Variability in soil characteristics and spatial rainfall amounts are known

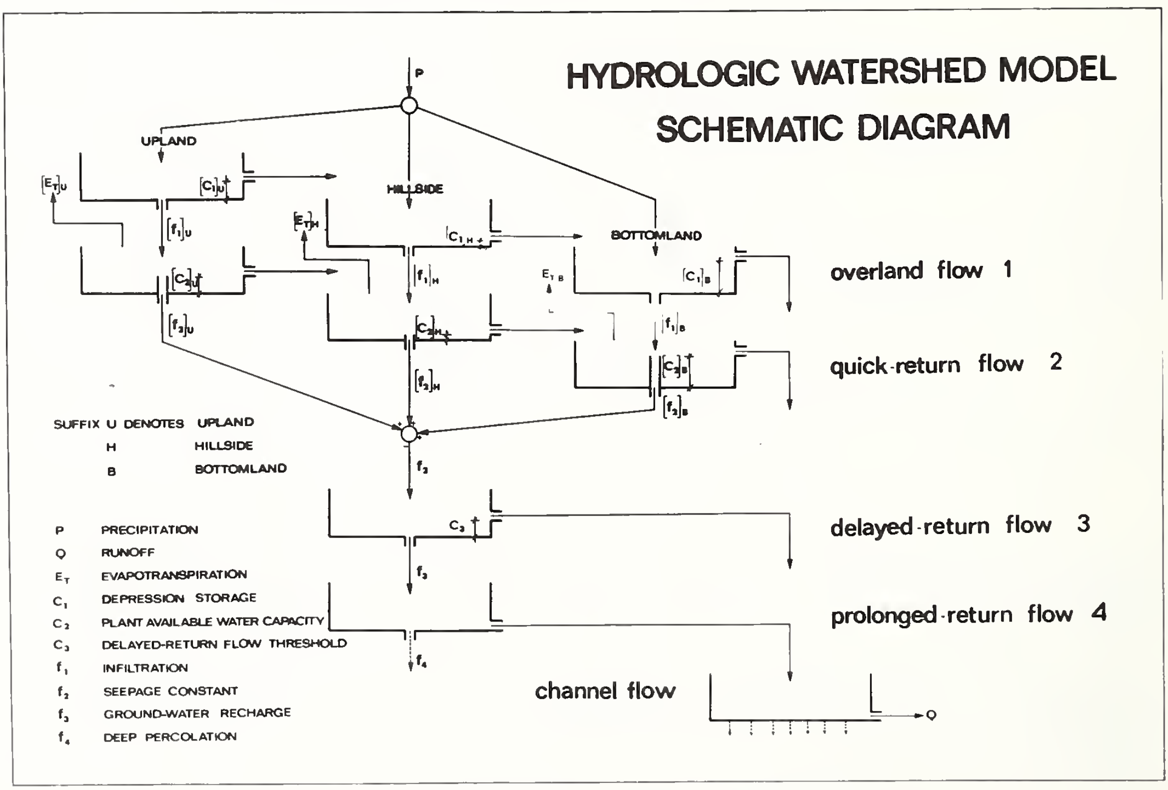


Figure 5.—Hydrologic watershed model—schematic diagram.

to generate differing amounts of precipitation excess over the watershed's surface. However, in hydrologic models, some degree of lumping is inevitable to make computation feasible. England and Holtan[3] postulated that there is a relationship between soil-type and elevation sequence. They suggested that a minimum of three hydrologic response zones are necessary—upland, hillside, and bottomland. The soils within each zone can be lumped for hydrologic purposes.

Sequential isolation of the three zones was accomplished by examining maps of land slope, degree of erosion, and soil-type. The schematic diagram (fig. 5) depicts the three response zones and their varying characteristics. Soil and vegetation parameters together with measured antecedent soil moisture were taken as fixed while the remaining threshold value and other unknowns, required a search procedure. For each flow regime, the corresponding storage coefficient, derived from hydrograph analysis, was used. If f(t) is the input and q(t) the output, the two basic equations for a single linear reservoir are:

$$S = K_q \qquad \text{(outflow equation)} \qquad (8)$$

and

$$f - q = \frac{d_s}{dt} \qquad \text{(continuity equation)} \qquad (9)$$

the outflow from the second reservoir is found by routing the outflow from the first reservoir through the second.

The complete model has been programed for an IBM 1620 computer. After routing, the various flow regimes are chronologically superimposed and summated for comparison with the total observed hydrograph.

Figures 6 and 7 compare the total observed hydrographs with the model's response to

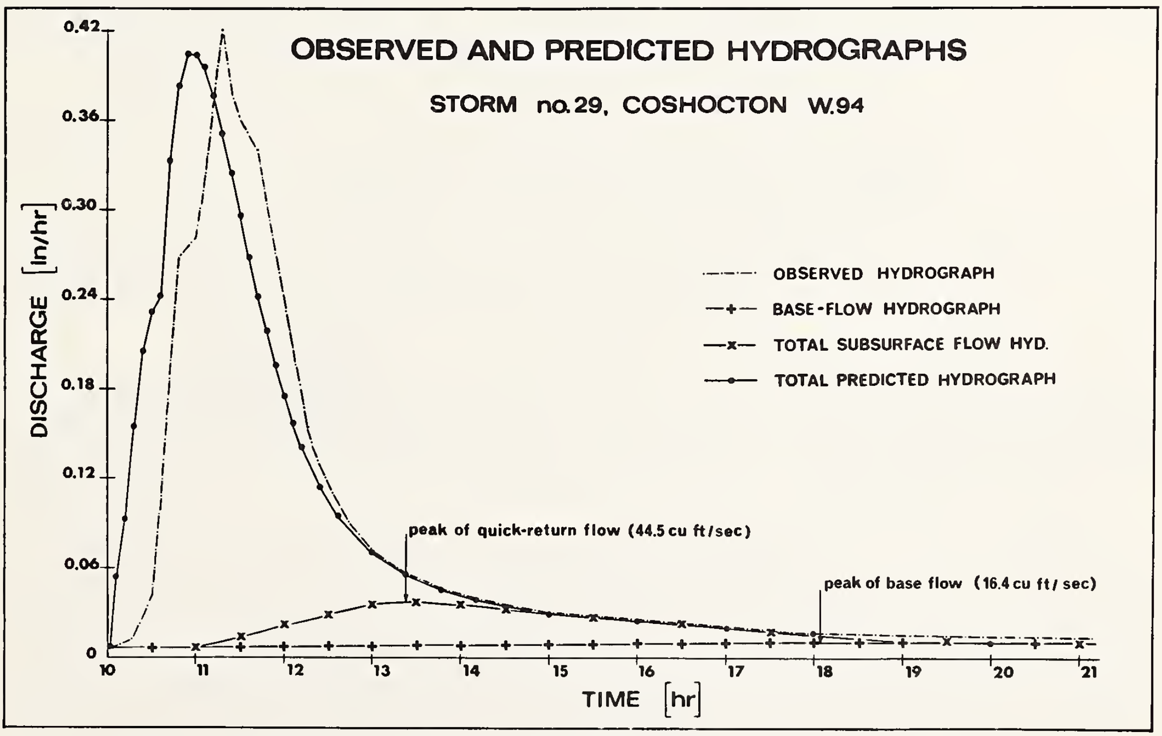


Figure 6.—Observed and predicted hydrographs—Storm No. 29, Coshocton, Ohio.

[3] England, C. B., and Holtan, H. N. Geomorphic Grouping of Soils in Watershed Engineering. 1967. Paper presented at American Society of Agronomy, Washington, D. C., November 1967.

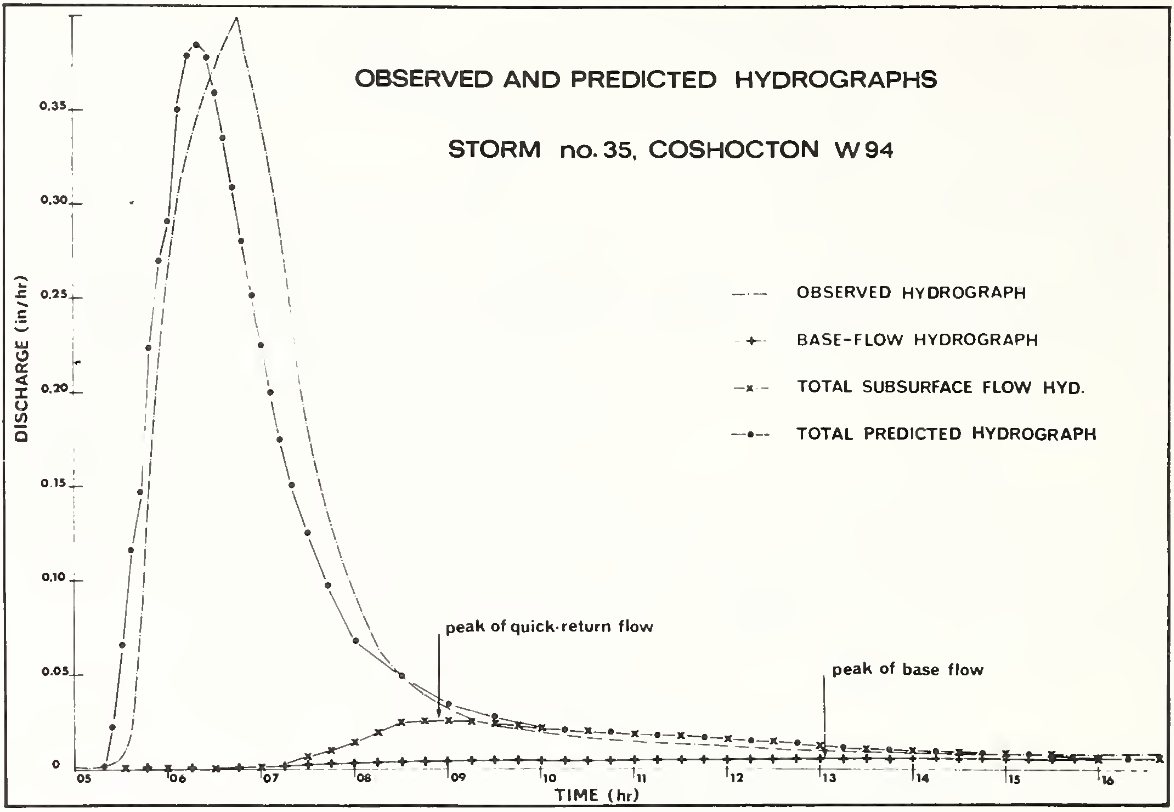


Figure 7.—Observed and predicted hydrographs—Storm No. 35, Coshocton, Ohio.

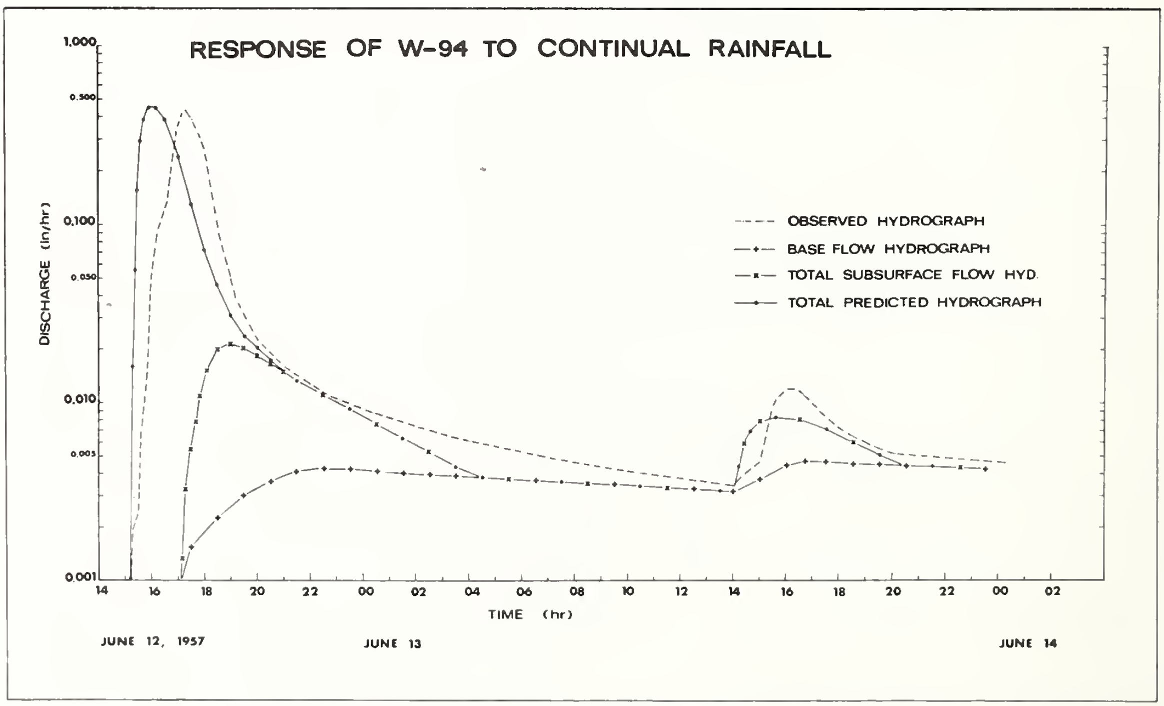


Figure 8.—Response of W-94 to continual rainfall—June 12-14, 1957.

the corresponding rainfall event. The prolonged and delayed-return flows were drawn as a single response and designated baseflow. Quick-return flow was calculated and summed with the baseflow. For completion, overland flow was added to facilitate comparison with the observed hydrograph. Figures 8 and 9 depict the model response to rainfall inputs

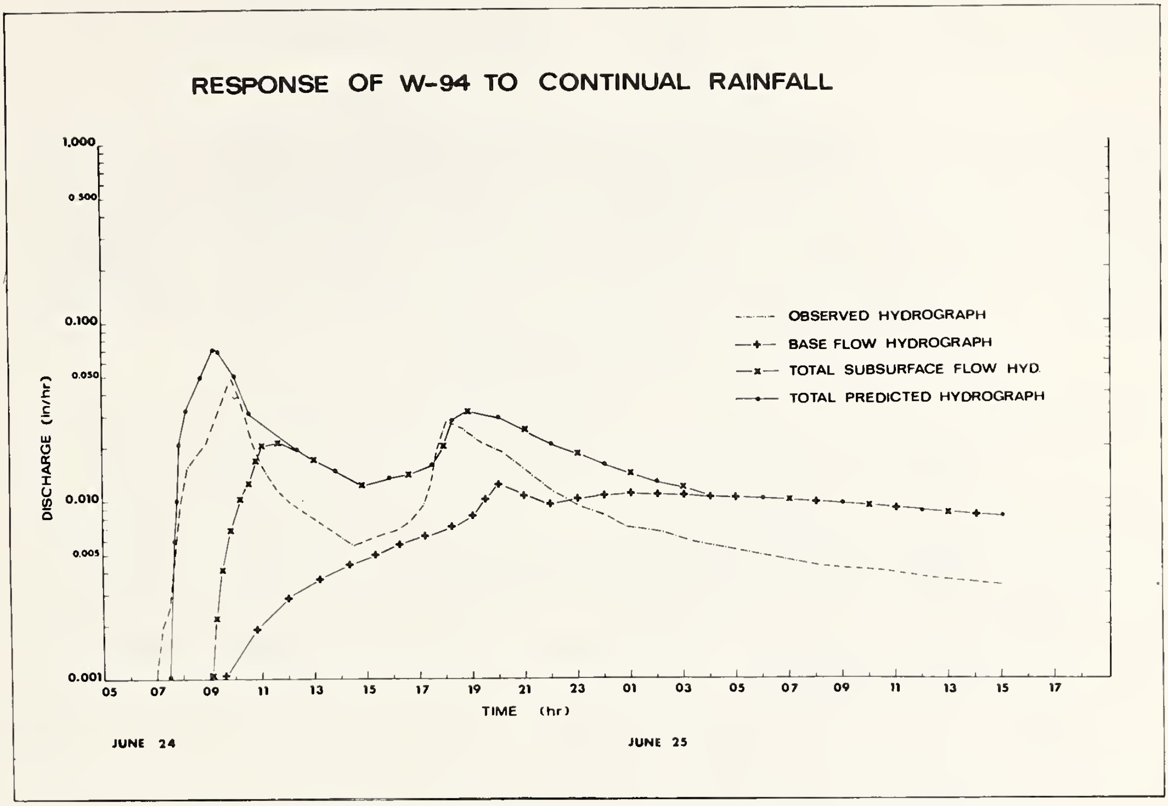


Figure 9.—Response of W-94 to continual rainfall—June 24-25, 1957.

from June 11-25, 1957. The response was plotted on semilogarithmic paper simply to amplify the subsurface flows. Because space was restricted, the plot was confined to periods of rapid variations in output (June 12-14 and June 24-25).

CONCLUSIONS

Historically, the separation of baseflow from the total hydrograph is quite arbitrary and, consequently, depends upon various concepts. In the past, this subjectivity was acceptable for flood events because baseflow was relatively unimportant. However, such methods evidently are no longer tolerable since the advent of more sophisticated simulation techniques.

Examples of these arbitrary methods are numerous, but the major limitations can be shown by the following. Linsley, Kohler, and Paulhus (*7*) describe the time lapse between the peaks of surface and baseflow in terms of "too short, too long, and about right." Brater (*1*) assumed the subsurface flow to commence at a time coincident with the surface runoff hydrograph and to attain a maximum value at the point of overland flow cessation. Hertzler (*3*) approximated the baseflow division to a straight line joining the start of hydrograph rise and a point on the recession curve where all flow is assumed to be baseflow. Snyder (*9*) extended the recession curve from the previous event downward to a point chronologically coincident with the peak surface flow. A convenient increment of time was selected during which baseflow was replenished before the recession reached a point where all the flow was baseflow.

Without exception, all the methods of baseflow separation terminate at a nebulous

point on the hydrograph's recession limb where all the flow is assumed to be baseflow. In addition, most make the assumption that the peak baseflow is somehow connected to an identifiable point on the surface runoff hydrograph. The former assumption is subjective; the latter is usually unrealistic. In contrast, the model presented here is an attempt to objectively simulate the various subsurface flow regimes. Despite the necessity of inferring the inputs to the various flow regimes, the model is realistic since it utilizes only the measured response of each flow regime.

However, the purpose of the model is not merely to separate baseflow from the total hydrograph but to simulate and later synthesize the land phase of the hydrologic system. The model is simple; although it possesses thresholds which make the system nonlinear, linear reservoir theory is assumed applicable above the thresholds. Few parameters need to be derived by iteration since the majority are inserted as measurable quantities. The system is flexible—reservoirs can be added, subtracted, or combined depending upon the response of the particular watershed. The simulation has only been demonstrated at Coshocton, Ohio, and therefore, requires further testing in differing physiographic areas.

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SYMBOLS USED IN TEXT

f capacity infiltration rate
f_1 actual infiltration rate
f_2 constant rate of seepage through first impeding layer

f_3	recharge rate of saturated zone
q_1	overland flow rate
q_2	quick-return flow rate
q_3	delayed-return flow rate
q_4	prolonged-return flow rate
K_1	storage coefficient of overland flow reservoir
K_2	storage coefficient of quick-return flow reservoir
K_3	storage coefficient of delayed-return flow reservoir
K_4	storage coefficient of prolonged-return flow reservoir
K_c	storage coefficient of channel network
S_1	storage status of overland flow reservoir
S_2	storage status of quick-return flow reservoir
S_3	storage status of delayed-return flow reservoir
S_4	storage status of prolonged-return flow reservoir
S_a	storage currently available above first restricting horizon
C_1	overland flow threshold: depression storage
C_2	quick-return flow threshold: plant available water capacity
C_3	delayed-return flow threshold
a	surface penetration index
$\triangle t$	time increment, determined by rainfall breakpoint intervals
q_r	any discharge on the recession limb of the complete system's hydrograph
S_r	storage status corresponding to discharge q_r

DISCUSSION

SESSION II A — HYDROLOGY AND SEEPAGE

Nazeer Ahmed: DelManzo has computed return flows assuming linear flows and found better results by Maasland method compared to the Glover method. If nonlinear flow is assumed and if Glover's method is assumed valid, could it be that return flow computation is better than the actual record?

D. DelManzo: The Glover and the Maasland method of return flow computation are both based upon the assumption that the existing nonlinear flows are small and can be neglected. If the nonlinear flows are significant, then the governing equation is nonlinear, and a solution other than that of Glover or Maasland must be used. For South Platte, a knowledge of the basin indicated that the assumption of negligible nonlinear flows was a reasonable one, and hence, either the method of Glover or Maasland could be applied.

The selection of which method to use is based solely upon the idealization of the pattern of water application to the land. The Glover method assumes an instantaneous application at one or more times during the month. The Maasland method assumes a constant rate of infiltration throughout the month. Because irrigation applications and precipitation occur in a random pattern throughout the valley, and because deep percolation tends to occur over a delayed period after application, the Maasland method was felt to be a closer approximation to the physical case than the Glover method. This was verified in a comparison of the two methods by a computer analysis for the South Platte.

R. Glover: Comment to D. D. DelManzo. Comments that in the South Platte Valley, which was the subject of DelManzo's paper of Session IIA, prevention of seepage would have serious effects on downstream users who would lose their supply. The operations of an individual in a highly organized society may have unintentional adverse effects on other members of the group. The possibility of such adverse interaction should be considered so that they may be prevented.

L. Myers: Mr. Glover's comment is valid and the effect of seepage reduction on other water users should certainly be considered. We should not assume, however, that a basically undesirable situation should be preserved because existing practice has been adapted to it. Downstream water users can benefit by the reduction of seepage losses from upstream diversions.

Water seeping from conveyance channels accumulates salt as it moves through soil on its way back to the river. The salt content of the Sevier River in Utah increases twentyfold along a 200-mile reach as the result of seven cycles of diversion and return flow. Most of this salt increase may come from leaching of irrigated lands, but an appreciable and unnecessary part of it is caused by seepage from conveyance channels. Reduction of seepage-caused salinity would unquestionably benefit downstream water users.

Evaluation of problems involving return flow from seepage is a complex process that must consider ground-water-flow patterns, soil salinity, evapotranspiration, leaching requirements, and other factors. Regardless of the complexities, we can be sure that sometime in the future, upstream irrigation water users will be allowed to divert only that quantity of water required for efficient application to their irrigated lands. It cannot be reasonably argued that the process of excess diversion, seepage loss, and return flow constitutes a desirable method of managing irrigation water supplies.

SESSION II B.— SEEPAGE CONTROL

Chairman: K. K. Barnes

IN-SITU-FABRICATED MEMBRANES[1]

Herbert Goldstein and Samuel I. Horowitz[2]

INTRODUCTION

The purpose of this talk is to present a new and unique system that permits the formation of a virtually impermeable membrane in situ by spray application, at ambient temperatures, of two low viscosity liquids. These two liquids are mixed immediately before application. As will be shown, application is done with commercially available equipment by an operator needing only moderate skill and a small amount of training. Application may be made over either dry or wet substrates, as long as no free water is present. The substrate need not be perfectly level.

The system is proprietary; patent applications are pending so I cannot reveal original compositions.

So that you may fully understand the background of the work and the line of reasoning that has led to a successful conclusion, I will first briefly deal with some history of asphalt-rubber combinations.

HISTORY OF ASPHALT-RUBBER COMBINATIONS

Rubber in Asphalt

The earliest references available for the use of rubber in asphalt are found in two British patents issued in 1843. Thomas Hancock who independently discovered the vulcanization of rubber by sulfur was issued one of these patents.

Most of the literature described these products for use as sealants, calks, and mastics. More recently, however, rubber has been used in paving asphalt. There were individuals who received patents for rubberized paving asphalt, which was tried by a few companies. Rubber manufacturers did not seek new markets until the 1930's, however, when the price of rubber was extremely low. The Dutch and British producers conducted studies laying rubberized asphalt roads in Holland, Java, Malaya, and England. Following World War II, one road section in Holland was found to have withstood the advance and retreat of the German military machine.

Large asphalt producers, such as petroleum refiners, have not shown much interest. However, their research departments more than likely have performed considerable research and development in this area.

Why is it desirable to put rubber in asphalt? The addition of rubber improves many of the properties of the asphalt. We find greater resistance to flow and increased elasticity and toughness. There is less brittleness at low temperatures, which is of great importance in the northern half of this country. There is greater resistance to aging, giving longer wear on roads, roofs, and so forth. The ability of rubber to absorb shock and vibration is especially important on roadways.

The major problem in adding rubber to asphalt is that the dispersed rubber must

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[1] Contribution from the Pioneer Division, Witco Chemical, New York, N. Y.

[2] Vice president, Pioneer Division, and chemist, Research and Development Laboratory, respectively.

swell to a certain extent to give the desired properties. Complete dispersion, however, does not occur, since rubber may agglomerate.

Because of the difficulty in producing uniformity, a maximum of 3 to 5 percent and, rarely, up to 10 percent rubber is used.

Asphalt in Rubber

Asphalt is used mainly as a plasticizer in rubber formulations. The products used are highly polymerized asphalts or "hard hydrocarbons." In general, the incompatibility of asphalt in elastomers can be attributed to the strong hydrocarbon nature of the asphalt. This has previously precluded any large usage of anything resembling 50:50 mixtures of asphalt and rubber.

DEVELOPMENT

Let us now discuss a bitumen-elastomer combination wherein the elastomer is formed by a chemical reaction directly in the bitumen. This reaction appears to be the same as in the asphalt-rubber combinations. However, the key phrase in bitumen-elastomer combination is "formed by a chemical reaction directly in the bitumen." This combination was not previously performed successfully. The general incompatibility of most rubber in asphalts and asphalt in rubbers have precluded such a combination. By the reaction that forms the elastomer, an infinitely dispersed polymer is now present in the asphalt. There is no necessity to swell the dispersed rubber as is required for a low percentage rubber in asphalt mix. There are two forms of composition. They differ by quantity in the composition and by quality in the finished product.

The low percentages of elastomer in asphalt yield thermoplastic products which, by their small amounts of polymer, can be described as rubberized asphalts. Yet, their nature has been changed. They are not asphalts, since the polymer imparts all the desired characteristics of rubberized material. They have greater flexibility at low temperatures; have resistance to flow; show properties of elasticity, toughness, ductility; and have greater abrasion resistance. The major difference from standard rubberized asphalts is the infinite dispersion of the elastomers throughout the asphalt in the system.

The other form of composition is referred to as thermosetting. These products are unmeltable, containing high percentages of polymer ranging from 30 to 90 percent. They combine the rubberlike properties of elastomers and the waterproofing and aging properties of asphalts.

CHARACTERISTICS OF INITIAL MATERIAL

We have developed in the last few years a variety of thermosetting systems. Some of these are 100 percent solids and others are filled systems. Relevant to the subject matter being discussed at this symposium is a sprayable solvent-extended system. The product system is made up of two components, both black, with densities at 77 F. of 7.7 lb./gal. for component A and 7.9 lb./gal. for component B, as shown below. The solids or nonvolatile content is 69 percent by weight for component A and 62 percent for component B. The mixing ratio is 1 to 1 with a solids content of 65 to 66 percent by weight. The viscosities of the two components are fairly low: 900 centipoises (c.p.s.) Brookfield for component A and 600 c.p.s. for B. The solvents used in this system are toluene and mineral spirits.

Characteristics of initial material

Typical Properties	Component A		Component B
Color	Black		Black
Density @ 77 F. lb./gal.	7.7		7.9
Nonvolatile content	69%		62%
Average - 65.5 percent			
Viscosity, Brookfield	900 c.p.s.		600 c.p.s.
Mixing ratio		1:1	
Diluents	Toluene & mineral spirits		Toluene & mineral spirits

CHARACTERISTICS OF FILM

Ambient temperatures are recommended as the curing temperature, as shown below. While higher temperatures accelerate removing the solvent and curing the elastomer, too high a temperature may skin the film before most of the solvent evaporates.

Characteristics of film

Typical Properties	
Gel time @ 77 F.	10-15 minutes
Set time @ 77 F.	Less than 4 hours
90 percent cure @ 77 F.	Within 24 hours
Physical appearance	Flexible black "rubber"
Tensile strength (ASTM D-412)	600 p.s.i. minimum
Elongation	250 percent
Durometer hardness — Shore A (ASTM D-2240)	70
Flexibility @ —30 F. (bend over 1-inch mandrel)	No cracking

The gel time at 77 F. is rather quick, 10-15 minutes with a set time of less than 4 hours. The film is 90 percent cured within 24 hours.

This film appears as a flexible black sheet that conforms to the contour of the substrate. The tensile strength is 600 p.s.i. minimum with a 250 per cent elongation. The film has a hardness of 70 on the Shore A scale and is flexible without cracking to —30 F. when bent over a 1-in. mandrel.

Antioxidant, built into the system, has performed well in the sunlight for over 11 months. We, also, have a small outdoor pond at our Perth Amboy laboratory that was coated in June 1966. This pond has been kept continuously full with a saturated salt solution and has lost water only through evaporation. We also coated a fish pond on Long Island that had a rather sandy porous soil. This pond has held water for over 8 months. The last time I visited the site, the fish were getting quite large. Previously, the pond was coated with asphalt which cracked continuously and as a result the pond could not hold the water.

The resistance of the film to various conditions was studied.

Under conditions shown as follows, there has been little or no loss of physical properties of the film. While we do not recommend using this composition as a coating for tanks containing sulfuric acid, we know that the film is unaffected by 40 percent sulfuric acid under spillage conditions.

Resistance of film

No loss of physical properties after exposure to the following conditions:

Conditions	Time	Temperature
Saturated salt solution	30 hours	Boiling point
Water	30 hours	Boiling point
40 percent sulfuric acid	7 days	150 F.
85 percent phosphoric acid	3 months	77 F.
50 percent caustic soda	3 months	77 F.
62 percent ammonium nitrate solution	3 months	120 F.
Petrolatum	3 months	120 F.
Saturated sodium chloride solution	21 months	Outdoor exposure (New Jersey)

MISCELLANEOUS

Ease of Application

Plastic films and synthetic rubber sheets have been on the market for some time. There are two major disadvantages. One is the requirement that sheets of some of these materials be sealed by hand. The second is the high cost of labor involved in laying out these sheets. Another, sometimes important, disadvantage is that the sheets may not conform to the substrate completely and, therefore, certain stresses may effectively be built into the lining.

The two-componet solvent extended material is sprayable through two-component metering equipment, such as those made by the established manufacturers of spray equipment. The necessity of using two-component spray equipment can be immediately realized when the gel time of 10-15 minutes is contemplated. With a spray tip of 25 mils or larger, we recommend a rate of application of 6.5- to 7-gal./100 sq. ft. This is equivalent to a cured film thickness of 1/16 inch. Equipment would normally be flushed with solvent, usually toluene or xylene. The ketone solvents, MEK and MIBK, and methylene chloride are even more effective.

For small applications the product may be brushed on, but due to its rapid gellation only small amounts of the two components should be mixed at a time. Because this is a liquid, it can conform to whatever the shape of the substrate.

Other Products

We have two other bitumen-elastomer products of interest to this symposium. Both are still in the development stage. One is a 100 percent nonvolatile filled system to be used for joint filling. The other is a variation which may be sprayed on a surface of water to form a tough, flexible, floating film.

RECENT ADVANCES IN PIPELINE TECHNOLOGY[1]

W. Allen Porter[2]

INTRODUCTION

In this highly advanced technological age, it is not unusual for concepts developed in one specialized industry to find widespread application far afield. Even so, it seems rather unlikely that such diverse disciplines as aerospace propulsion and water and sewage conveyance would find much in common. Strangely enough, however, this was precisely the case with the development of reinforced plastic mortar pipe, the subject of this paper.

Reinforced plastic mortar (RPM) pipe was developed by United Technology Center, Division of United Aircraft Corporation, as an outgrowth of its experience with reinforced plastic structures for flight hardware. It was designed specifically to meet the needs for improved water and waste conveyance systems for agricultural and domestic use. The data reported herein were generated for TECHITE®, UTC's RPM pipe.

As its generic name implies, RPM pipe is a composite material made up of fiberglass reinforcement in a synthetic resin-sand mortar.

This material received rapid acceptance in the past 2 years since its introduction on the market and promises to find wide use in irrigation, sewerage, and municipal and industrial water applications.

The following discussion is an introduction and general description of the RPM pipe and its capabilities.

MATERIALS AND MANUFACTURE

Reinforced plastic mortar pipe is constructed from three basic raw materials. Continuous strand glass filament serves as the reinforcing constituent and provides the major load-carrying capability. These filaments are produced by drawing molten borosilicate glass through orifices to a nominal diameter of 0.00055 in. and collecting them into groups called rovings, which are gathered and packaged on spools. Minimum specified tensile strength of the glass filaments is 200,000 p.s.i. The matrix is composed of an isophthalic-type polyester thermosetting resin and a graded stand aggregate. These materials, of course, are not new. They have been in wide use in structural applications for many years and their individual properties have been thoroughly characterized. When combined in a properly engineered composite, they yield structures that exhibit high strength and are lightweight, with a high degree of chemical inertness.

RPM pipe is manufactured by a process that was originally developed and refined by the aerospace industry for structural hardware, such as rocket motor casings and pressure vessels. Through suitable modifications of this process, large scale production of pipe products was possible on a truly economical basis. The process for RPM pipe consists of applying the glass filament reinforcement over a rotating mandrel in a fashion that orients them along the directions of the principal applied loads. The catalyzed liquid resin is added to form the matrix and the graded sand aggregate is introduced to provide the required bulk factor. Sufficient layers are added until the desired wall thickness is obtained. Bell and spigot sealing surfaces are molded integrally with the pipe. The result is a structure that has the required circumferential reinforcement to withstand internal pressure, longitudinal filaments for bending loads and sufficient wall thickness to impart the necessary rigidity for resistance to external burial loads. The

[1]Contribution from Techite Department, United Technology Center, Division United Aircraft Corporation.

[2]Product Manager, Techite Department, United Technology Center, Sunnyvale, Calif.

finished pipe section is oven heated while still on the mandrel to cure the thermosetting polyester resin. After the curing process is completed, which normally takes 30-45 minutes, the mandrel is extracted.

RPM PIPE CONFIGURATIONS

Reinforced plastic mortar pipe is produced in a wide variety of designs intended to serve the needs of agricultural, municipal, and industrial markets. Classes of RPM pipe range from gravity and low-head pipes for sanitary sewer applications to 200 p.s.i. pressure pipe for cross-country transmission and municipal water supply. Pipe designs of 60, 100, and 150 p.s.i. are also available, as well as special designs for service above 200 p.s.i. where they are required. Standard lengths are 10 and 20 feet, and diameters range from 8 to 48 inches. Standard joint configuration is a bell and spigot using a fully contained round rubber gasket as the sealing device. Figure 1 shows the RPM pipe joint.

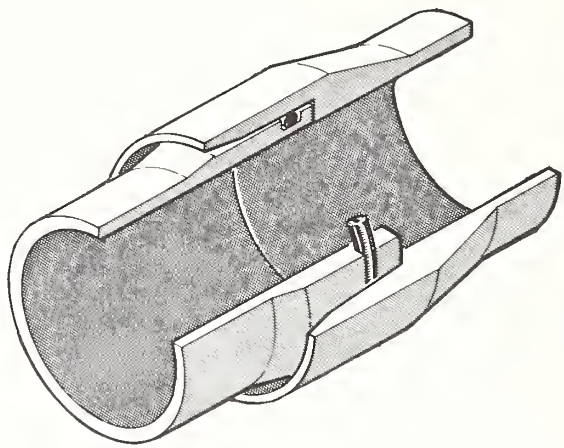


Figure 1.—Reinforced Plastic Mortar pipe joint.

TABLE 1. — Weights of Reinforced Plastic Mortar pipe

Diameter (inches)	Sewer	Pressure			Irrigation			
		Class 100	Class 150	Class 200	Class 60	Class 100	Class 150	Class 200
8	6.5	6.0	6.0	5.5	6.0	6.0	6.0	6.0
10	8.0	7.0	7.0	6.5	8.0	8.0	8.0	8.0
12	10.0	8.0	8.0	7.0	9.5	9.5	9.5	9.5
14	—	11.0	10.5	9.0	11.0	11.0	11.0	10.5
15	14.5	12.0	11.5	10.0	13.5	13.5	12.5	12.0
16	—	13.0	11.5	11.0	14.5	14.5	13.0	13.0
18	19.5	16.5	15.0	13.5	16.5	16.5	15.0	13.5
21	25.5	18.0	16.0	14.5	19.0	19.0	17.0	16.0
24	32.5	22.0	19.5	17.5	22.0	22.0	19.5	17.5
27	40.0	26.0	23.0	21.0	28.5	28.5	26.0	24.0
30	49.5	36.0	30.0	28.0	34.5	34.5	30.0	28.0
33	59.0	43.5	38.0	32.5	43.0	43.0	38.0	32.5
36	65.0	48.0	43.0	37.5	50.0	48.5	43.0	37.5
39	75.0	58.0	52.0	46.0	59.5	58.0	52.0	46.0
42	87.0	67.5	58.0	52.0	69.5	67.5	58.0	53.0
45	105.0	78.0	68.5	58.0	80.0	78.0	69.0	58.0
48	124.0	85.0	75.0	69.5	91.0	85.0	75.0	70.0

NOTE: The nominal weights of the higher classes of pressure and irrigation pipe are generally less than the weights of the lower classes for a given size of pipe. This is due to the substitution of fiberglass and resin for sand as the pressure class increases. The thickness of the pipe wall remains approximately the same.

Reinforced plastic mortar pipes are inherently lightweight, which makes them extremely attractive from an installation standpoint. Weights for typical types of RPM pipes are shown in table 1.

ENGINEERING PROPERTIES

Since reinforced plastic mortar is a composite material, the reaction of the pipe structure as a whole to imposed loads interests engineers, rather than that of any single constituent. For this reason, RPM pipe is generally specified on a performance basis in much the same way as asbestos cement pipe. Typical specifications call for hydrostatic tests, dimensional inspection, and verification of wall thickness or EI values. Mechanical properties for a typical RPM pipe design are as follows:

Class 60 Irrigation Pipe

Working pressure	60 p.s.i.
Burst pressure	475 p.s.i.
Hoop modulus	2.5×10^6 p.s.i.
Coefficient of thermal expansion	15×10^{-6} in./in./° F.
Coefficient of thermal conductivity	0.3 BTU/hr./ft.^{2}/°F./in.
Hardness (Barcol)	50-60
Specific gravity	1.9

An additional benefit to the engineer and user are the high factors of safety afforded by RPM pipe. For example, a Class 60 irrigation pipe will have an ultimate value for internal pressure between 6 and 8 times as great as its designated operating pressure. This occurs because of the inherent high tensile strength of the glass filaments. As sufficient reinforcement is added to the pipe wall to satisfy other structural criteria (the EI factor), hoop stresses due to internal pressure become quite low. In other words, internal pressure is not the governing design criterion for these pipes and the result is a high margin of safety to failure.

HYDRAULICS

The manufacture of RPM pipe over a polished steel mandrel produces an extremely smooth inner surface. This property and the close tolerances afforded by the concentrically molded joints account for the low friction losses exhibited by these pipes. RPM pipes are classified as hydraulically smooth, exhibiting flow characteristics that fall along the hydraulically smooth line on the Moody diagram. The use of the Moody diagram for RPM pipe is discussed in detail in recent studies performed by the Hydro-Science Corporation.

Although the use of empirical formulas such as the Manning or Hazen-Williams equations have questionable value when applied to this flow regime, they continue to be widely used for determining approximate head losses due to friction. An "n" value of 0.009 for the Manning equation and a "C_w" of 150 for the Hazen-Williams coefficient are commonly applied to RPM pipe. Solutions to flow problems for RPM pipe appear as appendix A.

Exceptionally good erosion resistance is displayed by reinforced plastic mortar materials due in large part to their hardness. For this reason, RPM pipes can be expected to maintain their high flow characteristics throughout their lifetime. In laboratory tests where pipes of different materials were subjected to scouring by slurries of sand and gravel simulating up to 50 years of normal service, RPM pipes showed negligible erosion.

CHEMICAL PROPERTIES

The materials of construction of RPM pipe are essentially inert in environments normally associated with water and waste conveyance. This chemical inactivity is reflected over a pH range from 1 to 10 at temperatures from minus 60 F. to plus 170 F. While RPM materials utilizing the isophthalic-type polyester resins are not recommended for continuous service with highly concentrated bases or at temperatures in excess of

170 F., other synthetic resins are available for extraordinary applications. Since reinforced plastic mortars utilize no metallic components, they are not subject to galvanic or electrolytic attack.

DESIGN CONSIDERATIONS

RPM pipes are classified as flexible conduits by virtue of their ability to deflect under external loads and to accept support from the surrounding soil. In this sense, they are analyzed in much the same fashion as corrugated metal or steel pipes. They differ from these types of pipes, however, in that they do not exhibit yield characteristics nor do they fail by buckling. RPM pipes may be deflected diametrically by as much as 20 percent of their original shape and will return to round when unloaded. Failure may be expected to occur at excessive deformation in an unsupported pipe by delamination caused by interlaminar shear stresses accompanied by tensile or bending failures of the outer fibers. From this it can be seen that the Spangler and Marston equations are useful for analyzing RPM pipe reactions when buried. An analytical treatment of design methods for RPM pipe appears in appendix B. Accepted practice for RPM pipe is to limit deflection to 5 percent of its original diameter. By utilizing proper installation techniques which provide adequate side support to the pipe in trench conditions, very high earth covers may be borne safely.

TEST PROGRAMS AND SPECIFICATIONS

In addition to the tests performed under the sponsorship of United Technology Center, several agencies have conducted or are conducting independent tests. Two test programs of particular significance are the National Sanitation Foundation (NSF) and the U.S. Bureau of Reclamation (USBR).

The National Sanitation Foundation Testing Laboratory completed physical and toxicological testing and evaluation of RPM pipe for potable water systems. TECHITE® RPM pipe is approved and authorized to use the NSF seal of approval.

The U.S. Bureau of Reclamation is conducting a 2-year test program on RPM pipe in sizes 12 through 96 inches. Principal areas of investigation are hydraulics, structural strength, and durability. The USBR laboratory in Denver recently completed tests on soil reaction simulating embankment conditions on 18-inch-diameter RPM pipe. At a soil pressure of 100 p.s.i., the pipe showed a deflection of approximately 40 percent while remaining stable. Structural tests of 96-inch-diameter pipe will begin this spring at the Denver facility.

Draft specifications for RPM pipe have been written by ASTM and are currently being reviewed. The program definition phase of Underwriters Laboratories has been completed and testing at their facilities will begin shortly.

A comprehensive study of RPM pipe in the buried state was completed by the soils engineering firm of Gribaldo, Jacobs, Jones, and Associates of Mountain View, Calif. The test installation involved 12- and 24-inch diameter pipes, trench depths of 10 and 20 feet and backfill compactions from 80 to 90 percent. Complete test results have been published; they are available from the author upon request.

APPLICATIONS

RPM pipe has found ready acceptance in a wide variety of applications in both municipal and agricultural markets.

During the past year when RPM pipe was available on a limited pilot plant basis only, over 80,000 feet of this material was installed in diameters ranging from 8 to 48 inches. Installations included gravity and force main sanitary sewers, irrigation distribution systems, industrial waste outfalls, and cross-country transmission including inverted siphons.

Two recent applications of RPM pipe deserve special mention. The first was the installation of some 5,300 feet of 27-in. diameter and 36-in. diameter thin wall RPM pipe inside existing failed concrete irrigation mains. The RPM pipe was produced with

special internally projecting rubber gasketed joints that provided a constant outside diameter only slightly smaller than the inside diameter of the concrete pipe. These liners, drawn into the concrete pipes from pits at the ends of the pipeline, were drawn in as much as 2,000 feet at one time. This technique shows significant promise for large-scale repair of failed or leaking pipelines because of its simplicity and low cost. This work, done by an irrigation district in California, resulted in substantial cost savings over replacement of the failed pipe and has provided permanent rehabilitation of the system. A detailed report of this project is also available from the author.

The second installation of note is the conversion of an existing open canal to a closed conduit system by placing lightweight RPM pipe directly in the canal bottom and backfilling to a height of approximately 3 feet over the top of the pipe. In this application, a 39-in. diameter pipe was chosen to deliver the required 40 second feet. This project in northeast Montana is being watched closely by many agencies who are interested in the trend from open- to closed-conveyance systems.

ECONOMICS

RPM pipe was developed to meet the requirements of high reliability and increased performance at a cost which would enable it to compete in the marketplace with currently used materials. Its ability to do so has been amply demonstrated in competitive bidding situations with asbestos-cement, reinforced concrete, vitrified clay, and lined and coated steel pipe. RPM pipe appears to become more attractive from an initial cost standpoint as size and working pressure increase. This is to say that its strongest competitive position is likely to be in sizes above 15-in. diameter where some hydrostatic head is to be encountered. In addition to competitive initial cost, RPM pipe presents substantial savings in installation costs due to its lightweight and superior handling characteristics. Because of its narrow profile, trenches for burying RPM pipes are narrower than for almost all other materials, resulting in excavation and backfill savings. Finally, because of its favorable hydraulic characteristics, RPM pipe may allow a smaller diameter pipe to be used than would be feasible with other materials.

FUTURE TRENDS

The future for reinforced plastic mortar pipe appears to be very bright indeed. As experience with RPM increases in the engineering community and as its performance and cost posture continues to be favorably demonstrated, its impact on the science of water conveyance will be substantial.

Increased automation in the manufacturing processes and improvements in both the cost and performance of raw materials promise even better things from RPM products.

The versatility of the material and the relative simplicity of its manufacture tend to open new areas of application. Among these, piping for the petrochemical industry both on and off shore, ground-water well casings, architectural columns, and slurry transmission systems represent a few of the most promising.

On-site fabrication of large diameter RPM pipes for aqueducts and penstocks has aroused considerable interest among planners for large future projects.

As these new materials and manufacturing techniques appear, the engineer will have many valuable tools to design efficient, economical, and reliable water systems. Reinforced plastic mortar pipe is an excellent example of recent advances in pipeline technology.

APPENDIX A—HYDRAULICS OF REINFORCED PLASTIC MORTAR PIPE

HYDRAULICS

To determine the flow conditions for water in a closed conduit, two formulas are widely used:

Manning formula

$$V = \frac{1.486}{n} r^{2/3} s^{1/2}$$

Hazen-Williams formula

$$V = 1.318 C_w r^{.63} s^{.54}$$

where: V = average velocity of fluid (f.p.s.)

r = hydraulic radius [ft. (D/4)]

s = hydraulic gradient (f.p.f.)

n = Kutter coefficient

C_w = Hazen-Williams coefficient.

The values for the coefficients "n" and "C_w" are a function of the friction properties of the internal hydraulic surface of the conduit and are arrived at empirically by means of tests and observations. The values of these coefficients for TECHITE® pipe have been determined to be $n = 0.009$ and $C_w = 150$. For design of a pipeline, more conservative values should be used to account for the normal joint gap and misalignment. Thus, the recommended safe design values for TECHITE® pipe in normal usage and service life are:

$$n = 0.010$$

$$C_w = 145$$

To aid in designing with TECHITE® pipe, graphs relating hydraulic gradient, fluid velocity, rate of flow, and pipe diameter for each method of computation are shown in figures 2 and 3. Figure 2 is based on the Manning formula using a value of $n = 0.010$. Figure 3 is based on the Hazen-Williams formula using a value of $C_w = 145$.

Given any two of the flow conditions, the other conditions can be determined from either graph.

Example: A pipeline with an average gradient of 2 ft. per 1,000 ft. has a minimum flow requirement of 6 second feet (c.f.s.). What diameter of TECHITE® pipe will be required to maintain this flow rate, and what will be the average velocity of the fluid?

Solution: Using the graph for the Manning formula (fig. 2), find the intersection of the gradient and flow lines for the given values. Read the nearest larger diameter of pipe required, in this example, 18 inches. Then, since the gradient is constant, follow the gradient line to the diameter selected and interpolate to arrive at an average fluid velocity of 3.6 f.p.s. (Note that the flow has increased to 6.3 second feet but the minimum is satisfied.)

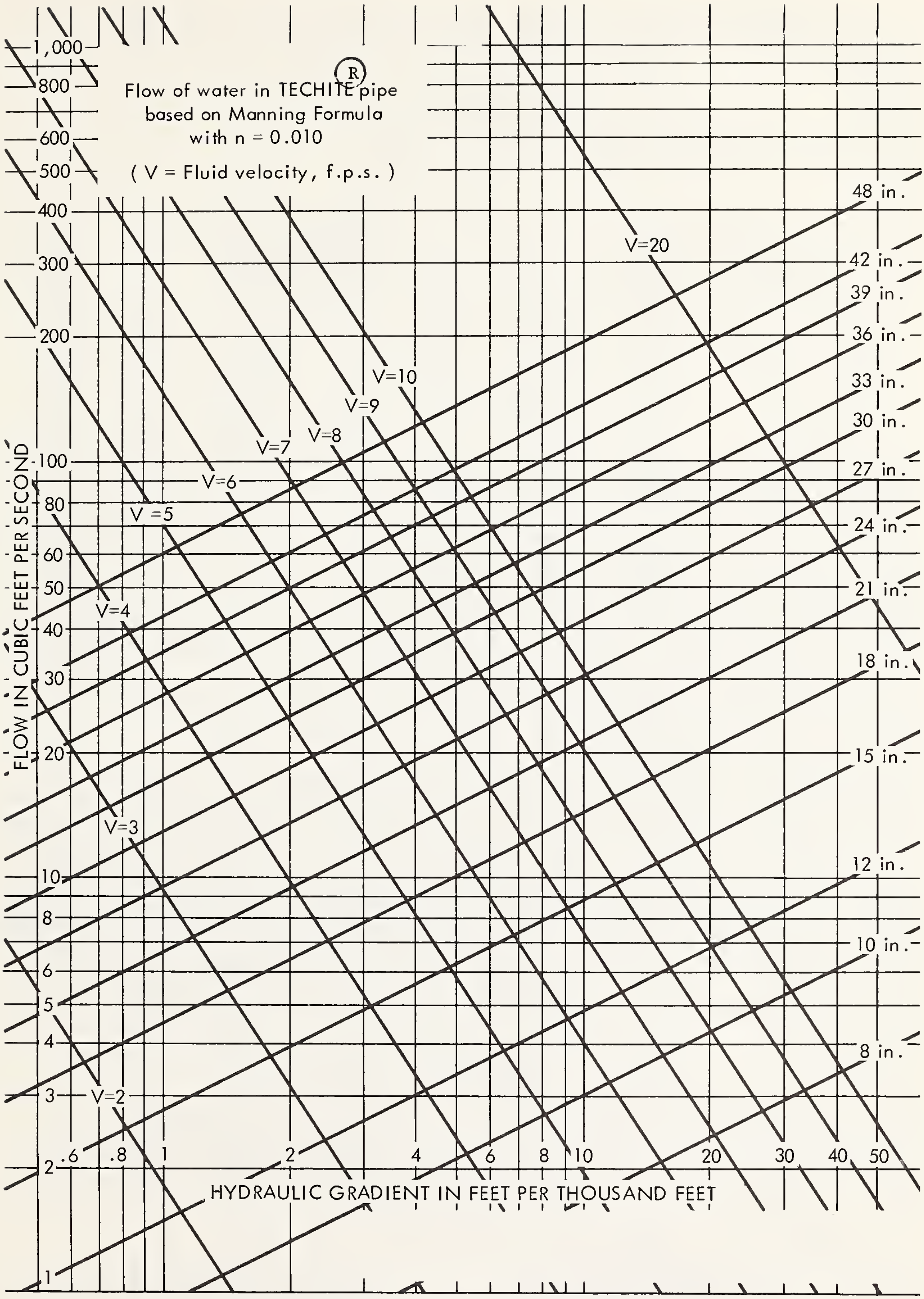


Figure 2.—Flow of water in TECHITE® pipe.

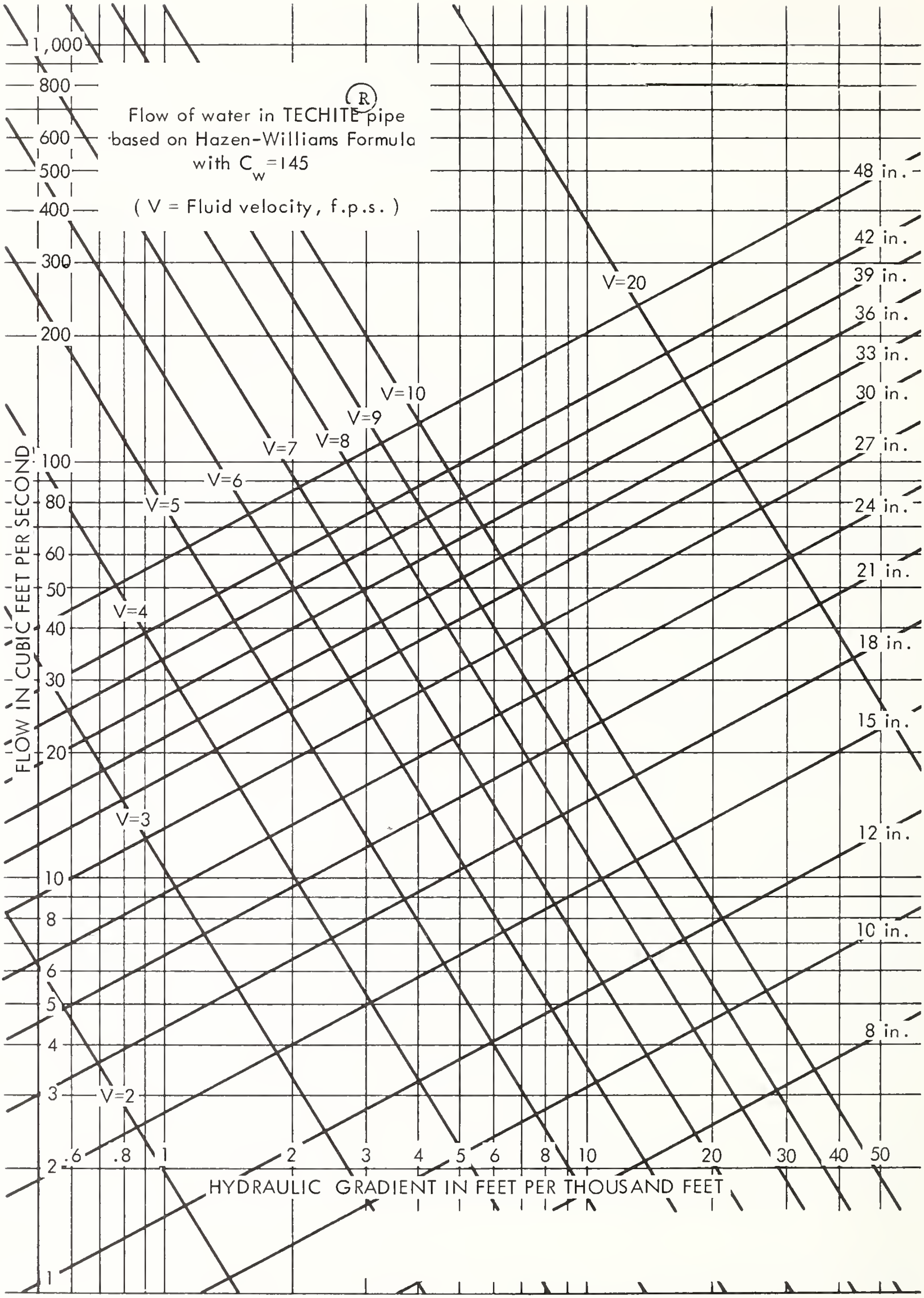
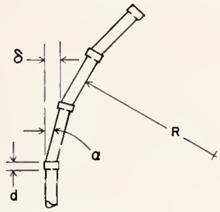


Figure 3.—Flow of water in TECHITE® pipe.

TABLE 2. — Irrigation and pressure pipe installation deflections



Size of pipe (in.)	10-foot section δ (in.)	α (degrees)	d (in.)	R (ft.)	20-foot section δ (in.)	α (degrees)	d (in.)	R (ft.)
8	6¼	3° 00′	⅜	190	12½	3° 00′	⅜	380
10	6¼	3° 00′	⅝	190	12½	3° 00′	⅝	380
12	6¼	3° 00′	¾	190	12½	3° 00′	¾	380
15	6¼	3° 00′	⅞	190	12½	3° 00′	⅞	380
18	6¼	3° 00′	1	190	12½	3° 00′	1	380
21	6¼	3° 00′	1¼	190	12½	3° 00′	1¼	380
24	6¼	3° 00′	1⅜	190	12½	3° 00′	1⅜	380
27	6¼	3° 00′	1½	190	12½	3° 00′	1½	380
30	6¼	3° 00′	1⅝	190	12½	3° 00′	1⅝	380
33	6¼	3° 00′	1⅞	190	12½	3° 00′	1⅞	380
36	6¼	3° 00′	2	190	12½	3° 00′	2	380
39	4¼	2° 00′	2	285	8½	2° 00′	2	570
42	4¼	2° 00′	2	285	8½	2° 00′	2	570
48	4¼	2° 00′	2	285	8½	2° 00′	2	570

NOTE: Standard straight TECHITE® pipe sections may be laid with deflections at each joint to accommodate minor changes in grade or alignment. A series of deflected joints may be used for curves provided the limits shown in the above table are not exceeded.

APPENDIX B. — DESIGN ANALYSIS FOR EXTERNAL LOADING OF REINFORCED PLASTIC MORTAR PIPE

IRRIGATION PIPE TRENCH LOAD ANALYSIS

The structural design of a buried conduit requires calculations of the probable maximum load, determination of the inherent strength of the pipe, and selection of a field or bedding condition that will insure that the supporting strength of the completed structure will be adequate.

Flexible pipe is defined as a closed conduit which can be distorted sufficiently to change its diameter more than 3 percent without causing harmful or potentially harmful cracks. TECHITE® pipe is properly classified as a flexible conduit and can sustain deflections up to 20 percent without damage.

The external loads on a buried flexible conduit are (1) earth load and (2) live load due to roadway traffic or heavy equipment crossings.

(1) The earth load in a trench condition may be determined by using Marston's equation:

$$W_c = C_d w B_c B_d$$

where: W_c = earth load on pipe (lb./lin. ft.)

C_d = load coefficient
B_c = outside diameter of pipe (ft.)
w = unit soil wt. (lb./cu. ft.)
B_d = trench width (ft.)

(2) Live loads may be calculated from the following formula recommended by the American Association of State Highway Officials:

$$W_1 = C_s \frac{PF}{L}$$

where: W_1 = concentrated live loads (lb./lin. ft.)

C_s = load coefficient
F = impact factor
P = concentrated load (lbs.)
L = effective length (ft.)

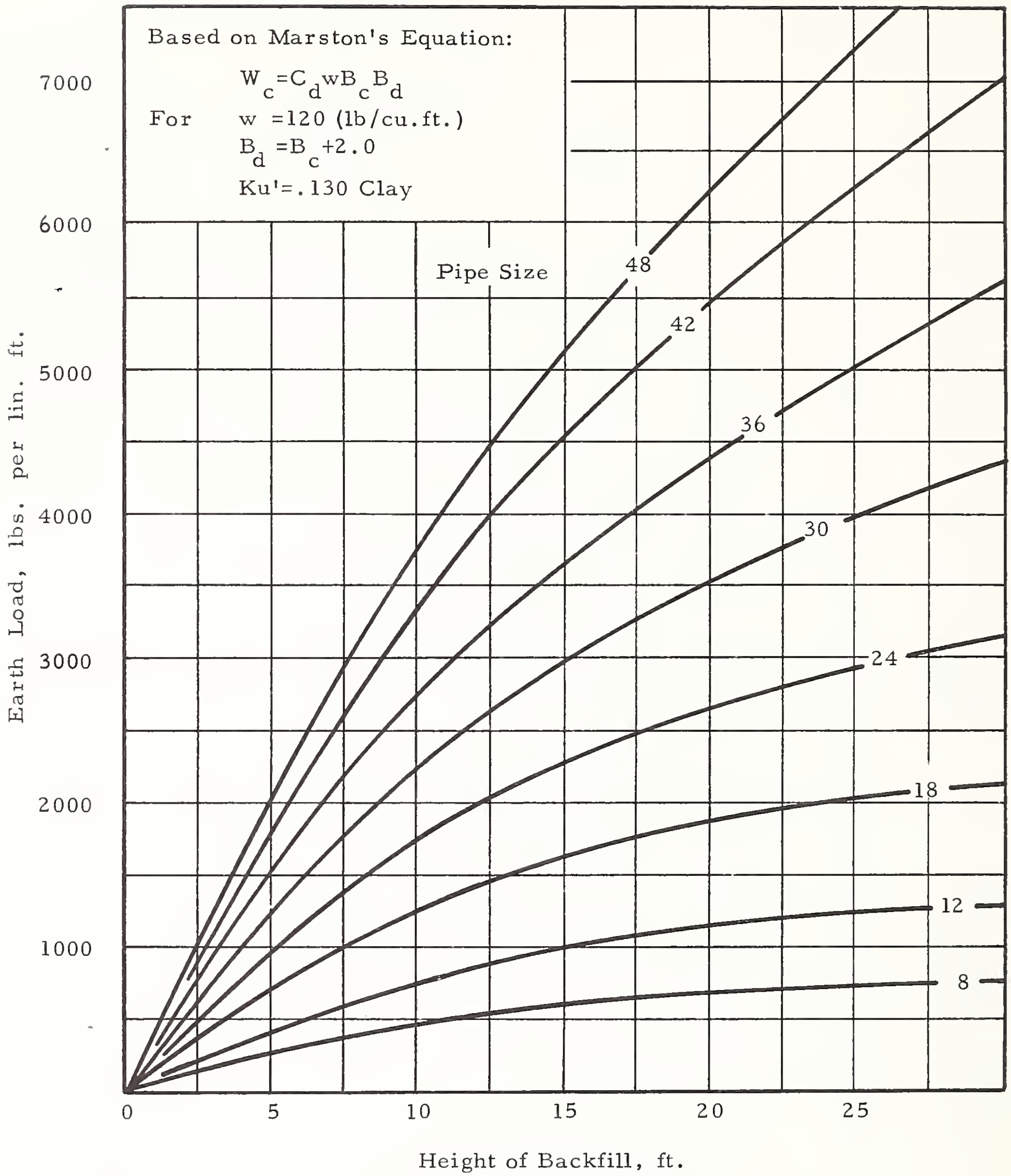


Figure 4.—Trench loads for TECHITE® irrigation pipe.

The following curves and table are presented as a guide to determine approximate loads, deflections, and a height of backfill. The earth loads and live loads for standard diameters of TECHITE® irrigation pipe under typical trench loading conditions are presented in figure 4 and table 3, respectively.

TABLE 3. — Live load vs. height of backfill (lb./lin. ft.)

Size of pipe (in.)	Height of backfill (ft.)									
	1	2	3	4	5	6	7	8	9	10
8	2,620	990	470	276	196	132	96	84	60	48
10	3,050	1,280	533	320	235	149	128	96	85	64
12	3,630	1,530	690	372	312	192	144	120	96	72
15	4,220	1,860	860	516	348	252	192	144	120	96
18	4,670	2,260	1,010	624	401	300	216	180	144	108
21	5,080	2,460	1,150	720	480	336	228	192	156	132
24	5,360	2,740	1,300	780	550	384	240	204	180	144
27	5,430	2,880	1,490	924	624	455	336	264	216	168
30	5,670	3,150	1,560	985	672	480	360	288	238	192
33	5,820	3,360	1,680	1,070	731	528	396	312	240	204
36	5,970	3,500	1,790	1,150	792	576	420	336	264	228
39	6,010	3,630	1,880	1,220	851	612	455	372	288	240
42	6,070	3,740	1,980	1,300	900	648	492	396	300	264
48	6,180	3,960	2,040	1,440	1,010	731	552	432	348	300

$W_1 = C_s \frac{PF}{L}$ where P = 16,000 lbs. and L = 3 ft. (for pipes longer than 3 ft.)
(Reference: American Association of State Highway Officials.)

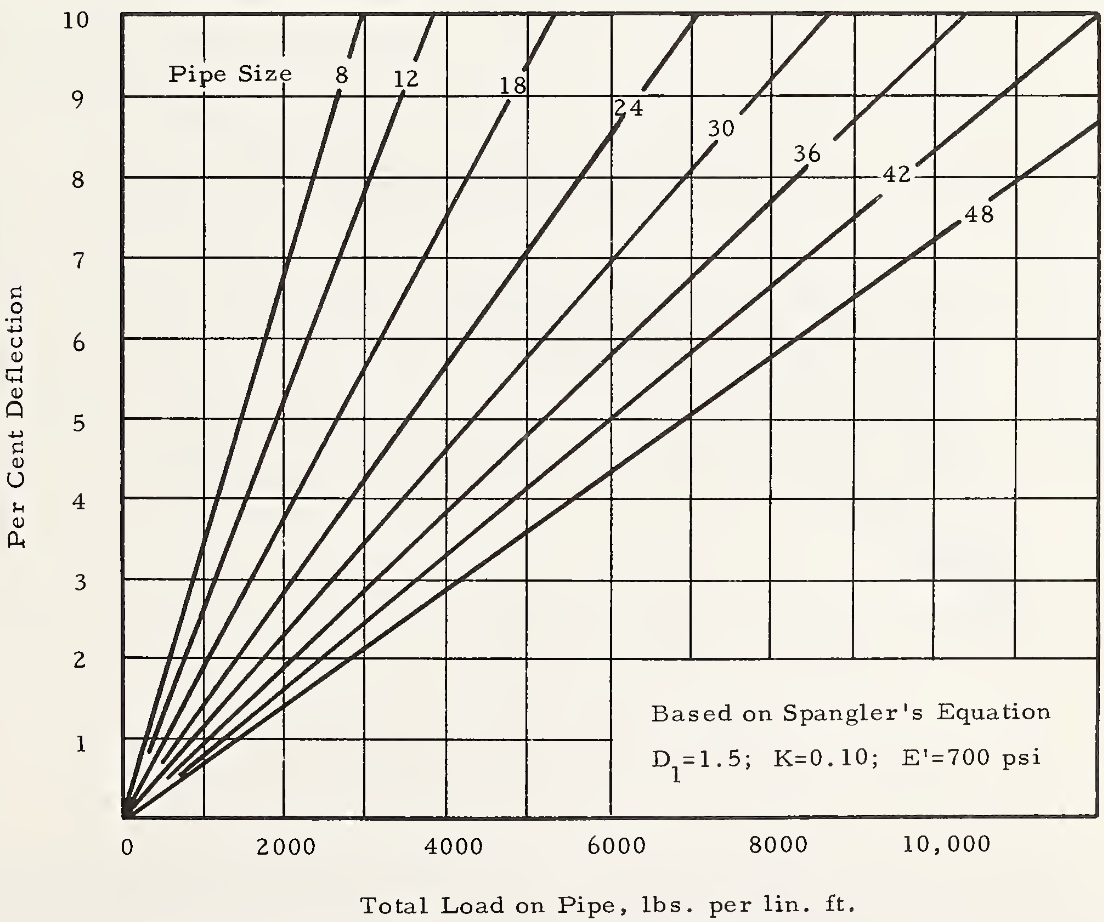


Figure 5.—Trench load vs. deflection for TECHITE® irrigation pipe.

Knowing the total load on the conduit ($W = W_c + W_1$), and the stiffness of the pipe, a bedding condition may be selected and the deflection of the pipe calculated by Spangler's equation:

$$\Delta x = D_1 \frac{KWr^3}{EI + 0.061E'r^3}$$

where: Δx = vertical deflection of pipe (in.)
D_1 = deflection lag factor
K = bedding constant
W = total load (lb./lin. in.)
r = mean pipe radius (in.)
EI = pipe stiffness factor (lb.-in.^{2}/lin. in.)
E' = soil modulus (lb./in.^{2})

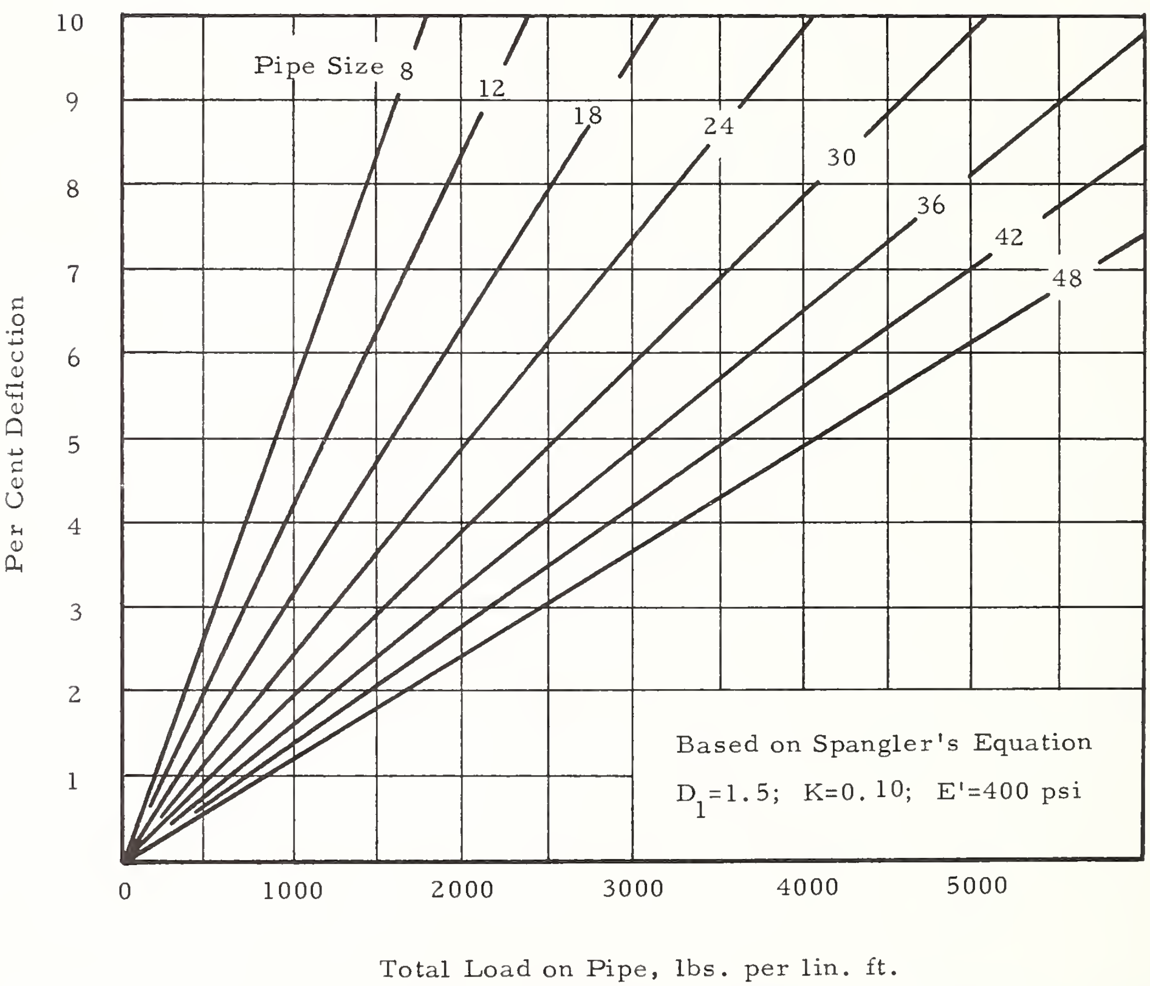


Figure 6.—Trench load vs. deflection for TECHITE® irrigation pipe.

Figures 5 and 6 present deflection vs. external load for standard sizes of TECHITE® irrigation pipe under typical conditions and for soil moduli of 700 and 400 p.s.i., respectively. The recommended permissible deflection is 5 percent. For design purposes, a factor of safety of 1.25 should be applied to the deflection.

The following examples are typical load analyses of irrigation pipe trenches:

Example 1. A 21-inch diameter irrigation pipeline is to be buried 5 feet in a clayey soil having an approximate density of 120 lb./cu. ft. and the soil compaction in the pipe zone is to be 85 percent. There is to be no vehicle traffic over the pipeline. What maximum deflection will be expected?

Solution:

a) Assume the trench width to be $B_c + 2.0$ feet and soil modulus (E′) to be 400 p.s.i.

b) From figure 4 the earth load is determined to be 800 lbs. per lin. ft.

c) The deflection is then determined from figure 6. For a load of 800 lbs. per lin. ft. on a 21-inch pipe the deflection is *2.2 percent.*

Example 2. Same conditions as example 1, except there will be vehicle traffic.

Solution:

a) Repeat steps (a) and (b) above.

b) From table 3 the live load is found to be 480 lbs. per lin. ft.

c) The total load (W) is the sum of the static and live loads. $W = W_c + W_1 = 800 + 480 = 1{,}280$ lbs. per lin. ft.

d) From figure 6, deflection is found to be *3.2 percent.*

Example 3. A 30-inch diameter pipe is to be buried in a clayey soil with a density of 110 lb. per cu. ft., and the soil compaction in the pipe zone is to be 90 percent. A bedding angle of 30 degrees is specified. What is the maximum height of backfill the pipe can withstand?

Solution:

a) Assume the trench width to be $B_c + 2.0$ ft. and the soil modulus (E′) to be 700 lb. per sq. in.

b) Assume a deflection of 5 percent with a 1.25 factor of safety as recommended. Therefore, the maximum deflection will be 4 percent.

c) From figure 5, the load at 4 percent deflection is determined to be 3,400 lb. per lin. ft.

d) The maximum height of backfill is obtained from figure 4. Since figure 4 is based on a soil density of 120 lb. per cu. ft. and the earth load is directly proportional to the density, a correction should be made to the earth load as follows: $W = (\frac{120}{110})\ 3{,}400 = 3{,}700$ lb. per lin. ft. Thus, the allowable height of the backfill is *22 ft.* The effect of the live loads is negligible at this depth. The minimum cover to protect the pipe from live loads as determined from table 3 will be approximately 2 ft.

DISCUSSION

SESSION II B — SEEPAGE CONTROL

H. R. Cedergren: Do you have any information on the cost of the film particularly in large installations?

H. Goldstein: At the present, we believe that the maximum cost for a 60-mil film using commercial equipment and commercial contractors will not exceed 25 cents per square foot. This cost can be reduced if lesser thicknesses are used and we assume that as the installation grows in size, the actual cost applied will decrease due to savings such as in labor.

D. A. Donohue: Do your membranes adhere to natural porous media and steel?

H. Goldstein: Adhesion is excellent not only to natural porous media and steel, but also to various plastics.

D. A. Donohue: What is the upper temperature limit before the membrane begins to decompose?

H. Goldstein: Decomposition may set in if the membrane is exposed to 300 F. or over for long periods of time. Transient exposure will not start decomposition below 350 F.

R. J. Bennett: Have you any long-term weathering data?

H. Goldstein: Actual exposures thus far have been for 2 years in sunlight, and weather-ometer tests indicate an absolute minimum of 3 years. Insofar as exposed surfaces, which have been protected with ceramic granules, we anticipate an indefinite life.

A. R. Dedrick: Do you anticipate any problems in controlling the thickness of the finished membrane when applying *in situ?*

H. Goldstein: We do not anticipate any serious problems inasmuch as the thickness of the membrane can be controlled by the rate of application of the liquid material to any given surface area. In actual practice, a skilled operator should be able to accurately control his coverage. Insofar as this skill is concerned, it can be acquired with one or two days practice using normal commercial equipment.

D. Wallace: What range of pipe diameters do you manufacture?

W. A. Porter: Reinforced plastic mortar pipe is presently being manufactured in diameters from 8 to 48 inches. Production capability for sizes up to 96-in. diameter is planned by late 1968.

D. Wallace: What tools are used in tapping or slotting pipe?

W. A. Porter: Conventional tools with carbide-tipped bits are normally used for penetrating RPM pipes. Cutting may be accomplished with masonry saws.

M. E. Jensen: Do you have any data on the change in hydraulic roughness due to abrasion by sand or sharp gravel in the water?

W. A. Porter: Accelerated abrasion studies have been conducted on RPM pipe in which the pipe was subjected to wear by slurries of sand and gravel. The results of these tests showed no pitting and only a very slight, regular wearing of the surface. That is, the mean surface roughness showed negligible increase as measured by a profilometer. For hydraulically smooth pipes, we are concerned that the ratio of e, the mean surface roughness to δ, the boundary layer thickness, remains small. Since spalling or chipping of the surface does not occur and since there are no gross changes in surface roughness due to abrasive action, we have concluded that the original hydraulic properties will essentially be retained throughout the lifetime of the pipe.

SESSION III — SEEPAGE CONTROL

Chairman: Amalio Gomez

CURRENT SEEPAGE REDUCTION RESEARCH[1]

Lloyd E. Myers and Robert J. Reginato[2]

INTRODUCTION

Seepage problems are highly variable, and no single corrective method or material can be applied in all situations. Since most of our seepage problems remain unsolved, we have not yet developed methods and materials that are truly effective, economical, and practical for many situations. Industrial, university, and governmental research workers are slowly but steadily improving our ability to control seepage. Some of the current work and progress is being discussed by other symposium speakers. This paper will briefly report four operational methods of seepage control developed by the U.S. Water Conservation Laboratory. These methods relate to the use of waterborne-asphalt emulsions, soil sealing with sodium carbonate, sprayable crack sealers, and asphalt-fiberglass linings.

WATERBORNE-ASPHALT EMULSIONS

A number of materials can be added to the water in small reservoirs to seal the soil and reduce seepage losses. We have found that certain cationic-asphalt emulsions can be used effectively for this purpose. Asphalt emulsion can be added to the water either as the reservoir is initially filled or after it is full. This latter method requires some means of mixing the emulsion with water. The asphalt moves with the water and mechanically plugs the soil pores. The emulsion is toxic to fish for several days after application.

Waterborne-asphalt emulsions have reduced seepage under a wide variety of conditions. For reasonable assurance of success, however, the following requirements should be met: (1) the asphalt emulsion should be stable and infinitely dilutable in water, (2) treated soils should be nonexpansive, (3) pretreatment seepage rates should exceed 1 foot per day, (4) weed growth in the pond should be eliminated, (5) mechanical damage to the seal should be prevented, and (6) water should be kept in the pond continuously. These criteria were developed through studies of 20 different asphalt emulsions, involving hundreds of laboratory tests and more than 50 field trials.

Our most recent pond-sealing experience with waterborne emulsion was in two reservoirs of 375,000-gallon capacity each that were constructed in sand and gravel. Pretreatment seepage loss was 3 feet per day for pond 1 and 2.53 feet per day for pond 2 when filled to about a 5-foot depth. Both ponds were treated by adding emulsion to the inflowing stream as the ponds were filled. Pond 1 was treated in September 1966 with 0.8 gallon emulsion per square yard of bottom and side surface. Pond 2 was treated in May 1967 with 1 gallon per square yard. In September 1967 the seepage rate had declined to 0.03 foot per day for both ponds. This is a seepage reduction of about 99 percent. Both ponds are now being used to raise fish and the treatments appear to be highly successful.

Pond sealing with waterborne-asphalt emulsions presently costs from $0.75 to $1.00 per square yard. This cost can easily be cut in half as the production of suitable emulsions becomes competitive. The requirements for successful seepage control with asphalt emulsions are somewhat restrictive, but there are many locations where they can be

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[1]Contribution from the Soil and Water Conservation Research Division, Agricultural Research Service, U.S. Department of Agriculture.

[2]Director and Research soil scientist, respectively, U.S. Water Conservation Laboratory, Phoenix, Ariz.

met. Ease of application, particularly in reservoirs that cannot be dewatered, makes waterborne-asphalt emulsions an attractive material for reducing seepage.

SODIUM CARBONATE

Seepage from many unlined reservoirs in calcium-aggregated soils can be greatly reduced by treatment with sodium carbonate without compaction. The sodium causes soil aggregates to disperse and swell and, consequently, to plug the water-conducting soil pores. Sodium salts are used rather widely for seepage control, but the salt usually recommended is sodium chloride or one of the several different forms of the sodium phosphates. Often the amount of salt applied is estimated, and heavy compaction of the treated soil has been thought necessary.

When we began to consider the use of sodium salts for sealing stock ponds in Arizona, we questioned these previous recommendations and procedures. Soil compaction with heavy equipment is not feasible in remote areas and application of too much sodium results in excessive dispersion so that the treated soil is easily eroded. There was also a question concerning the relative effectiveness of different sodium salts. Subsequent investigations resolved the issue in favor of sodium carbonate applied at rates determined by soil analysis and applied without soil compaction.

Soils involved in our studies contained calcium-aggregated montmorillonite clay, so that they were quite permeable. These soils are typical of the Southwestern United States. Field and laboratory studies included the use of sodium chloride tetrasodium pyrophosphate, and sodium carbonate. Sodium carbonate proved superior to the other salts. In addition to providing sodium for soil deflocculation, it ties up calcium as insoluable calcium carbonate and complexes. The other salts tie up calcium less effectively, if at all. Sodium chloride treatments of stock ponds failed in 6 months and a tetrasodium pyrophosphate treatment failed in 18 months. The sodium carbonate treatment described below is still effective 5 years after application.

House Mountain No. 1 is a livestock watering pond approximately 0.25 acre in size. It had never held water from the April runoff until the start of the grazing season in June. Pretreatment seepage was at least 2 inches per day and was more likely about 5 inches per day. The pond was built in a highly aggregated silty clay (51-percent clay, 47-percent silt, and 2-percent sand) with a cation exchange capacity of 55 meq./100 g. We decided to apply 2,000 pounds of sodium carbonate to create an exchangeable sodium percentage of 13 in the top 3 inches of soil. The dry pond was first cleared of weeds and large rocks, and the salt was scattered by hand over the soil surface. A grid of stakes and string was laid out and the proper amount of salt was applied to each grid area to aid in uniform application. The salt was then mixed with the soil with a small tractor and disk. This was done in July 1963 and the pond has held water ever since, except for periods of about 30 days each in September 1964 and 1967. Posttreatment seepage has been about 0.15 inch per day both before and after periods of soil drying.

Seepage increased to 0.3 inch per day in May 1966, and soil analysis showed that sodium was slowly being leached from the soil. In November 1966, 200 pounds of sodium carbonate and 300 pounds of sodium chloride were broadcast into the pond water from a boat. The amount of sodium carbonate was restricted to make sure the pH of the water stayed below 9.5 so that the water would not be unpalatable to cattle. Seepage declined to 0.15 inch per day by December 1966 and the soil sodium exchange percentage increased to the original posttreatment value.

Laboratory and field studies indicate that sodium carbonate can be used without compaction to reduce seepage from unlined ponds where the following conditions exist: (1) the depth of soil overlying sand, gravel, or porous rock is at least 12 inches, (2) soil clay content is 15 percent or greater, and (3) cation exchange capacity exceeds 15 meq./100 g. of soil. Treatment of the 0.25-acre House Mountain pond cost $80 for salt and about $140 for labor and equipment rental. Total cost was approximately $220 or $0.18 per square yard. Apparently, the treatment can be maintained indefinitely by occasionally adding relatively small quantities of salt to the water in the pond.

SPRAYABLE CRACK SEALERS

Seepage losses from concrete-lined canals and storage structures occur through cracks

in the concrete. Cracks not only lose water but contribute to progressive deterioration of the lining. Temperature changes cause the cracks to open and close. The cracks widen and deteriorate because of grinding and spalling of the edges as they close on sediment deposited while they are open. This type of lining deterioration can be prevented by applying a flexible surface seal over the cracks while they are still small. Crack sealing is not widely used, however, because commonly used methods involve laborious and expensive concrete cleaning procedures and applying the sealing materials by hand.

We have developed a simple, efficient method, utilizing commercially available power equipment, for cleaning and sealing cracks in concrete hydraulic structures. Cracks are easily and rapidly located and cleaned with a high pressure (400 p.s.i.) water jet provided by standard orchard-spray equipment. The water jet rapidly blasts sediment out of the cracks and removes silt and algae around their edges. Crack sealer is then sprayed directly on the wet concrete at a pressure of 1,000 p.s.i. using a small, light-weight, gasoline-powered diaphragm pump. Bonding of several sealants to wet concrete was evaluated, including two nonsprayable rubberized asphalt mastics specified for application to dry concrete only. All of the sealers, including the mastics, bonded well to clean, wet concrete. In one field test, three men cleaned and sealed 800 lineal feet of cracks in 1 hour with 10 gallons of sealer.

The feasibility of using power-spray equipment for cleaning concrete cracks and applying sealants has been proved. But a completely satisfactory sprayable sealer has not yet been found. Asphalt compounds have intimately bonded to clean, wet concrete and performed well for 2 years of field exposure, but longer life than this is usually required for the procedure to be economically feasible. Satisfactory sprayable sealers can be formulated as thixotropic emulsions of durable elastomeric compounds. We are now waiting for some commercial firm to become interested in the development of such a product.

ASPHALT-FIBERGLASS LININGS

Strong, impermeable reservoir linings can be fabricated in the field with fiberglass matting and asphalt emulsion. Prefabricated asphalt planking is widely used for reservoir lining, but it presents problems in shipping and handling. Laying jute fabric in ditches and reservoirs and spraying it with melted asphaltic cement has been used. However, this type of lining is unacceptable because the jute deteriorates. Many of the problems associated with prefabricated asphalt planking and with jute-asphalt cement linings can be eliminated by using fiberglass and asphalt emulsion.

Installation of asphalt-fiberglass linings is relatively simple. The ditch or reservoir sides and bottoms are shaped to be reasonably but not perfectly smooth. Compaction is desirable, but successful linings have been installed over uncompacted soil. Either asphalt emulsion, diluted 1 to 1 with water, or cutback asphalt is sprayed on the soil at a rate of about 0.5 gallon per square yard. Spraying is done with a gasoline-powered gear pump and cone-spray nozzles with the swirl-plates removed. Heating the emulsion to about 140 F. is desirable, but is not necessary if sufficient power is available for operating the pump with cold emulsion. Cutback asphalt must be heated. Unwoven fiberglass matting is laid over the sprayed soil in 50-inch-wide strips with about 5 inches of overlap between strips. The matting weighs about 9 ounces per square yard, is easily handled, and can be cut to size with scissors or shears. Leather gloves must be worn while handling the fiberglass. After the glass is laid, it is sprayed with undiluted emulsion at a rate of 0.3 to 0.5 gallon per square yard. Water in the emulsion dissolves the starch sizing in the fiberglass so that the mat conforms to irregularities in the soil surface. The lining should be allowed to cure for several days and then sprayed or mopped with about 0.15 gallon per square yard of asphalt-clay roofing emulsion.

A 165,000-gallon reservoir, with a lined area of 505 square yards, was lined in 1962 at a materials cost of $0.70 per square yard: $210 for fiberglass, $100 for asphalt emulsion, and $50 for asphalt-clay emulsion. Reservoir side slopes were 2 horizontal to 1 vertical and the lining was installed because the side slopes were unstable. Three men sprayed the asphalt base coat in 45 minutes, six men installed and sprayed the fiberglass in 4 hours, and two men mopped on the seal coat in about 6 hours. Since 1962 costs of materials have declined; now, they would total about $0.55 per square yard in the Phoenix, Ariz. area.

Asphalt-fiberglass linings are tough, impermeable, and durable. An experimental ditch lining of this material was trampled by cattle without damage, whereas adjacent test sections of asphalt-jute, plastic, and artificial rubber were severely damaged. Two asphalt-fiberglass-lined reservoirs in northern Arizona are in excellent condition after nearly 6 years of exposure.

SUMMARY

Four different seepage control methods were described. Waterborne-asphalt emulsions are effective provided certain requirements are met. They are easy to apply and may be particularly useful in leaky reservoirs that cannot be dewatered. Sodium carbonate is a low-cost treatment for reducing seepage from unlined reservoirs in calcium-aggregated soils, which are common in the Southwestern United States. Power-spray equipment was used successfully in cleaning cracked concrete linings and applying crack sealers with a significant reduction in labor costs. Field tests showed that effective linings can be fabricated with fiberglass and asphalt emulsion. These linings are simple to install, relatively low in cost, and are strong and durable. Although additional improvements are possible and will be beneficial, these four methods and materials are useful in their present form. They should be considered by anyone confronted with a seepage problem.

CHARACTERISTICS OF THERMOPLASTIC AND ELASTOMERIC LINERS[1]

D. H. Moeller and J. R. Ryffel[2]

INTRODUCTION

The intent of this paper is to discuss briefly some basic features of high polymer structure and indicate how these features affect the properties of the various liner materials. We will describe how the basic structure of these giant molecules is related to the properties observed in waterproofing linings made from butyl rubber, polyvinyl chloride, polyethylene, and a relative newcomer, chlorinated polyethylene. We hope this discussion will help you to understand the advantages, disadvantages, and limitations of each type of lining material, and help you make the best engineering choice of material for your waterproofing and water conservation projects.

Giant molecules or high polymers, as the chemist calls them, are not new. Wood, wool, silk, cotton, starch, and meat are naturally occurring high polymers that have been with us for a long time. However, there are many synthetically produced polymers being used in sheet and film form as liners that have been developed within the last 30 years.

Synthetic polymers may be hard or soft, brittle or tough, stiff or supple, leathery or rubbery, meltable or nonmeltable. These qualities may be explained, at least partly, in terms of the structure of the molecules and how they are arranged.

AMORPHOUS POLYMERS

Two basic structural categories into which high polymers can be classified are (1) amorphous, and (2) partly crystalline. A dictionary defines amorphous as "lacking in definite form or shape." Tobolsky[3] used an analogy in describing polymer structure that should be useful here. Amorphous polymers resemble a bowl of cooked spaghetti that has been violently stirred. Tremendously long molecular chains are coiled and twisted together at random. In their internal disorder they resemble liquids more than solids, and as a result, act in some ways like liquids.

Amorphous polymers as a group comprise glassy, leathery, and rubbery materials. The apparent differences in these materials are due to their different states at room temperature. Any of the amorphous high polymers can become glassy, depending on the temperature. Rubber can shatter like glass at about —160 F. The glassy amorphous plastics become leathery and then rubbery upon heating and finally, some melt down to a viscous liquid.

An amorphous polymer generally goes from a glassy through leathery to a rubbery state with a change of 80 degrees Fahrenheit. Each polymer has its own "glass-transition temperature" at which it changes from a glassy, brittle state to a leathery state where it is flexible but tough. This temperature is determined by the structure of the molecules and the energy binding the molecules together. It is sometimes called the "second order transition temperature" (which term, I believe, filled chemist's need to be mysterious).

These various stages from glassy, to leathery, to rubbery, to liquid can perhaps be further illustrated by our spaghetti analogy. Figure 1 shows the glassy state of a polymer in which the long molecular chains are frozen immobile. There is no movement except for the oscillation or vibration of the atomic groups represented by the small dots.

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[1] Contribution from the Dow Chemical Company.

[2] Head, Technical Service Section, Construction Materials Research and Development, Plastic Products Div., Dow Chemical Co., Midland, Michigan, and research supervisor, Research and Development Dept., Dow Chemical Co., Plaquemine, La., respectively.

[3] Tobolsky, A. V., The Mechanical Properties of Polymers. Scientific Amer. 197:120-134, illus. 1957.

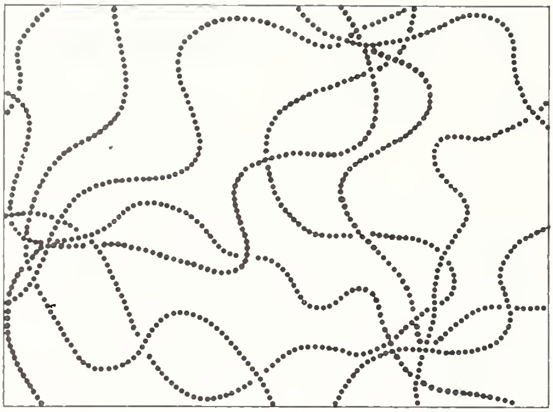


Figure 1.—Amorphous polymer, glassy state.

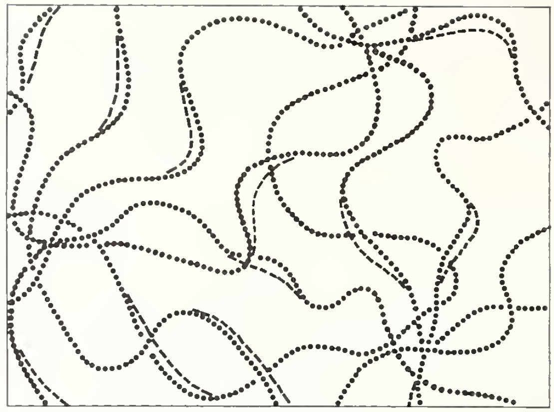


Figure 2.—Amorphous polymer, leathery state.

As the polymer is warmed, the molecular chains themselves begin disorderly thermal movements as represented in figure 2. The "spaghetti comes alive" — short segments of the molecular chains begin to move but are restrained by any cross-links present and by the entanglement and attraction forces of the molecular groups. At this stage the material is leathery — flexible but quite tough and boardy.

Further heating, figure 3, results in greater molecular chain movement and the material becomes less tough and more pliable and will become rubbery. Entire free lengths of the molecules between cross-links and centers of entanglement are in rapid motion, twisting and rippling.

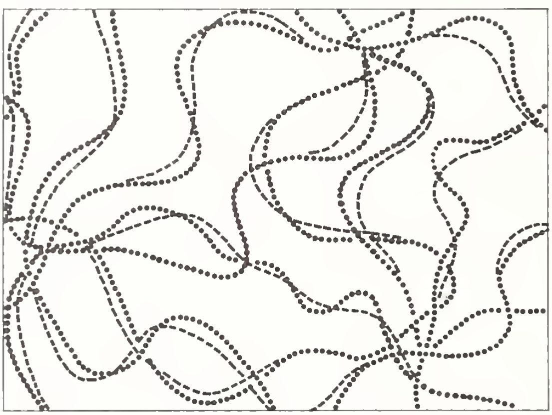


Figure 3.—Amorphous polymer, rubbery state.

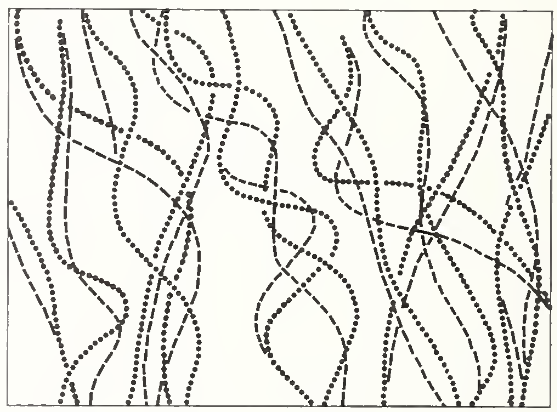


Figure 4.—Amorphous polymer, liquid-flow state.

Finally, figure 4, (a thermoplastic containing no permanent cross-links) the centers of entanglement become undone and the spaghetti chains, in addition to twisting and rippling, can slide past each other in viscous liquid flow. This can be as a result of a thermal stress or a long-term mechanical loading. If the movement is a result of the latter, we call it creep. In this area, polymers resemble viscous liquids.

As implied in the previous references to cross-links, the tendency to liquid flow at elevated temperatures or due to long-term mechanical stress can be eliminated if the molecular chain entanglements are made permanent by chemical cross-linking. This is what vulcanization does in a rubber or elastomer. Rubber unfreezes from the glassy state to the flexible state at relatively low temperatures because its molecules are inherently limber and are only weakly held together except at cross-linking points. They cannot melt because of the cross-linking. On the other hand, some polymers with stiff molecules and strong molecular attraction forces remain glassy until relatively high temperatures are reached, but then they successively become leathery, rubbery, and finally, melt because no cross-linking exists. Polystyrene would be an example of the latter type of

amorphous polymer. It has a glass transition temperature of approximately 212 F. compared with butyl rubber at approximately —94 F.

Table 1 gives approximate glass-transition temperatures of common polymers. These glass-transition temperatures can be difficult to determine, are dependent on the method of measurement, and the values are often the subject of debate. They do not represent equivalent or comparative rigidity in the various polymers, but merely represent a type of phase change within a given polymer.

TABLE 1. — Glass-transition temperatures[1]

Polymer	Temperature, T_G	
	°F.	°C.
Butyl rubber (cross-linked polyisobutylene)	—96	—71
Polyethylene (amorphous)	—13	—25
Chlorinated polyethylene (40 percent chlorine, amorphous)	—4	[2]—20
Plasticized polyvinyl chloride	—58 to 77	[2]—50 to 25
Unplasticized (rigid) polyvinyl chloride	180	82
Polystyrene	212	100

[1] Frisch, H. L., and Rogers, C. E. Transport in Polymers, Jour. Polymer Sci., Pt. C (12):297-315. 1966.

[2] By Dow test, torsion pendulum @ 1 c.p.s.

CRYSTALLINE POLYMERS

Now let's move from the amorphous polymers to the other broad classification — the partly crystalline polymers. Instead of being completely disordered or random, the molecular chains in crystalline polymers have a regularity or orderliness. This orderliness or crystallinity ranges up to 90 percent or more of the whole volume. Until about 1957, the spaghetti analogy was used to describe the partly crystalline polymers also. With the application of advanced analytical techniques to the study of polymer structure, the old "fringed micelle" concept (as it was called) was found to be fundamentally incorrect. It is still worthwhile, however, to use this old concept because it best illustrates the differences between amorphous and partly crystalline polymers.

Fringed Micelle Concept

In the fringed micelle concept, the orderly crystalline regions or micelles, as shown in figure 5, were believed to be formed when several molecular strands fell parallel to one another in a regular 3-dimensional lattice. The areas or fringes between lattices remain amorphous and disordered. The alinement of the molecular strands at the micelles was felt to account for the increased strength of partly crystalline polymers since the closely adjacent atomic groups (represented by the small dots) form strong bonds with

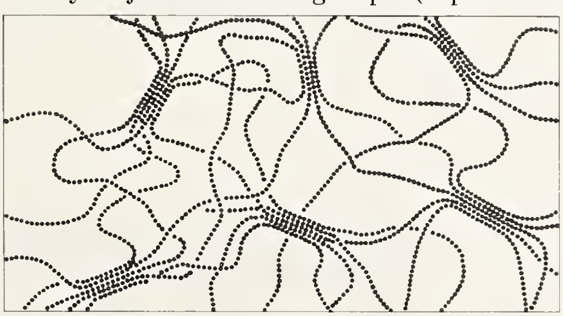


Figure 5.—Fringed micelle concept, partly crystalline polymer.

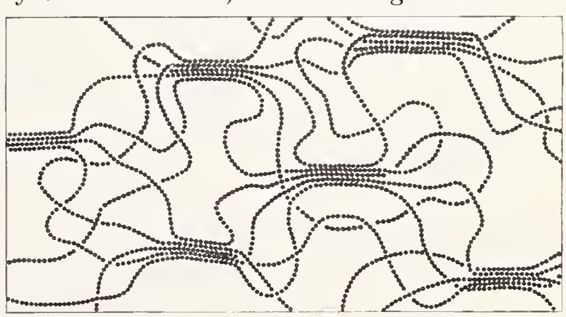


Figure 6.—Fringed micelle, oriented crystalline polymer.

each other. The micelles act like cross-links in limiting the amount of creep. Partly crystalline polymers have a glassy state which they leave when melting at a fairly definite temperature, but they generally do not become leathery or rubbery.

Figure 6 shows an oriented crystalline structure where the micelles are lined up in the same direction. Their alinement increases the strength in that direction. This orientation is determined largely by the method of fabrication. In general, orientation increases the strength of the polymer in the direction of the orientation but can also result in dimensional instability or shrinkage when the polymer is heated.

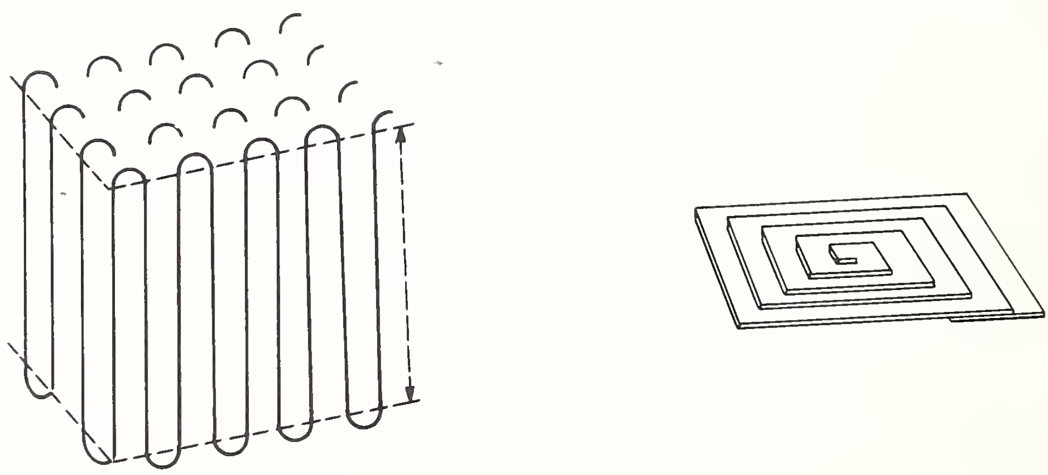
Partly crystalline polymers often maintain their rigidity and strength over a temperature span of several hundred degrees above the glass-transition point. The percentage of crystallinity tends to slowly decline as the temperature rises, then at a certain critical temperature all of the crystals melt and the polymer goes into a liquid phase. In general, crystalline polymers have higher temperatures of liquid flow than amorphous polymers (see reference listed in footnote 1).

Polyethylene and polyvinyl chloride are two partly crystalline polymers that are used as liners for water conservation. As shown on table 1, the glass-transition temperature of polyethylene at —13 F. is quite low and polyethylene provides the low-temperature flexibility and toughness needed for most liner applications. Unmodified polyvinyl chloride has a glass-transition temperature of 158 F.; therefore, at almost all liner-use temperatures it would be in its glassy, brittle state. This problem is resloved for liners by incorporating plasticizers that give the material the requisite flexibility and toughness.

At this point it might be helpful to look at the amorphous but cross-linked rubbers or elastomers. A degree of orderliness (crystallization, if you will) of the molecular chains can be achieved in some natural and synthetic rubbers by stretching. Stress crystallization in an elastomer can be easily seen by viewing the rubber molecule as one of our strands of spaghetti. As the polymer is stretched, the chains become elongated in the direction of stretch and chain elements become parallel and form essentially a crystalline lattice much like that shown on figure 6, except that the amorphous parts of the molecules are in a highly elongated state. Anyone who has stretched a rubber band has seen the reinforcing effect of stress crystallization.[4]

Lamella Concept

New tools such as the electron microscope and X-ray diffraction have enabled



a) A PORTION OF A REGULARLY FOLDED LAMELLAR CRYSTAL

b) GROWTH OF SINGLE CRYST. FROM SCREW DISLOCATION

Figure 7.—Crystalline polymer structure.

[4] Cooper, W. and Grace, N. S. Elastomers. Jour. Polymer Sci. Pt. C (12):133-150, illus. 1966.

scientists to look at these giant molecules as single-molecule crystals. As a result, the concept of partly crystalline structures has changed. Instead of many molecular chains meeting at micelles or crystallites, it is now believed that the giant molecular chains are regularly folded on themselves and that these folded areas or lamellae constitute the crystalline, or ordered part of polymers. Figure 7 illustrates what one such crystalline polymer structure might be like. The polymers crystallize in platelike crystals — the chains folding back on themselves when grown from the polymer melt or from solution.

POLYMER STRUCTURE EFFECT ON PROPERTIES

Now that we know a little bit about the structure of amorphous and crystalline polymers, what is the effect of structure on the physical and chemical properties we observe?

Mechanical Properties

We have already covered many effects of structure on mechanical properties; however, we can summarize and add to them.

Tensile Properties

Crystallinity has a strengthening effect although it generally reduces ultimate elongation. Orientation or alinement of the molecules can also increase strength in the direction of orientation but also reduces the ultimate elongation in that direction. Amorphous polymers, in their rubbery states above the glass-transition temperatures, have relatively low strength. Stress crystallization can increase strength of elastomeric materials when they are stretched.

High-Temperature Strength and Creep

Crystallinity has a major effect in increasing the high-temperature strength and reducing the creep of a polymer up to its specific melting point. Cross-links in plastics or elastomers also considerably enhance high-temperature tensile properties and reduce creep. Amorphous polymers, above the glass-transition temperature, have low resistance to creep.

Low-Temperature Performance

The glass-transition temperatures given in table 1 are really not too useful in comparing the low-temperature performance properties of liner materials. A better but still imperfect indicator of low-temperature physical performance of a polymer is the ASTM D 746 brittleness temperature given in table 2. This test involves striking small cantilevered specimens with an impactor that subjects the specimens to a combination of rapid impact and shear stresses. The temperature at which 50 percent of the specimens break or crack is defined as the ASTM brittleness temperature. The test is very thickness sensitive (the thinner the specimens, the lower the measured brittleness temperature) and, therefore, the thicknesses of the specimens are shown to better correlate the data. Although polyethylene is a crystalline material, it is seen to have inherently good low-temperature flexibility and impact properties that are not obvious from its glass-transition temperature.

TABLE 2. — ASTM D 746 brittleness temperatures

Liner material	ASTM brittleness temperature (°F.)	Thickness of test specimen (inch)
Butyl rubber	—40	0.060
Chlorinated polyethylene (SARALOY® 660)	—45	0.060
Polyethylene	—90	0.075
A plasticized polyvinyl chloride	— 3	0.060

Chemical Resistance

Generally speaking, crystallinity increases the resistance to chemical diffusion and softening. The more orderly the molecular chains are folded on themselves, the greater the resistance to chemical diffusion. Diffusion appears to take place at the imperfectly ordered or amorphous regions. Amorphous polymers as a class are therefore generally poorer in chemical resistance than crystalline polymers. Cross-linking also appears to increase chemical resistance, although in some rubbers the amount of residual unsaturation is also very important.

Solvent Welding and Adhesive Joining

While chemical resistance obviously can be a great advantage for containing chemicals, it can be a disadvantage when using adhesives or solvents to adhere or join polymer surfaces. Polyethylene does not lend itself to either adhesive joining or solvent welding. Butyl rubber has similar problems, although not to the same degree as polyethylene. Plasticized polyvinyl chloride (PVC), although crystalline and somewhat chemically resistant, can be joined with solvent-based adhesives. Chlorinated polyethylene (CPE), which is amorphous, can be welded easily with a solvent or joined with adhesives. Both CPE and PVC, although quite resistant to softening and attack by inorganic chemicals, are not effective barriers to most organic chemicals.

Dielectric and Heat Welding

If a polymer molecule is symmetrical as to the distribution of its atomic groups, the electrical charges are evenly distributed and the material is not polar. Although all plastics can be heated dielectrically in a rapidly rotating electrical field, only those that have a fair degree of polarity have a high enough loss factor to be heat welded by radio frequency dielectric heating. Polyethylene is not adaptable to dielectric welding since it is not polar. Butyl rubber cannot be heat welded by any means since it is cross-linked or vulcanized and therefore won't melt and fuse together. Polyvinyl chloride and chlorinated polyethylene are polar materials with high dielectric loss factors and can be, and frequently are, dielectrically welded into large liner tarpaulins.

Heat welding of polyethylene, polyvinyl chloride, and chlorinated polyethylene is possible by several other methods: Heated metal plates; hot-gas welding; and sonic welding. Partly crystalline polymers must be heated beyond their crystalline melting point to perform the welding, and this sometimes causes difficulties because of the narrow range of temperatures of proper melt flow.

Weatherability

To be resistant to weathering, polymers must have a saturated chemical structure; that is, no double bonds. Almost all depend also on the ultraviolet screening properties of carbon black additives or other ultraviolet absorbing chemicals. All of the liner materials commonly used are saturated; however, since polyvinyl-chloride liners depend on plasticizers for their flexibility, the weatherability of the vinyl liners can vary considerably depending on the plasticizers used. The presence of chlorine atoms or other certain chemical groups often tends to stabilize a molecular chain against ultraviolet degradation.

SUMMARY

To sum up, table 3 compares the structural, chemical, and miscellaneous properties of the four liner materials we discussed. It is hoped that this comparative chart and the discussion will help you make your selection of liner materials on a basis of greater knowledge and understanding.

TABLE 3. — Comparison of properties of specified liner materials

Property	Polyethylene	Polyvinyl chloride	Butyl rubber	Chlorinated polyethylene
Structure	Crystalline	Crystalline (plasticized)	Amorphous, cross linked	Amorphous
Tensile strength	High	Medium to high	Medium	Low
Ultimate elongation	Low to medium	Medium	Medium	High
Low temp. flexibility	Excellent	Fair to good	Excellent	Excellent
Chemical resistance	Excellent	Good	Good	Good
Solvent weldable	No	No	No	Yes
Heat weldable	Yes	Yes	No	Yes
Dielectrically weldable	No	Yes	No	Yes
Weatherability	Fair to good[1]	Varies	Excellent	Excellent
Cost[2]	3.5¢/sq. ft. of 8 mil	10¢/sq. ft. of .020″	20¢/sq. ft. of .031″	12¢/sq. ft. of .020″

[1] Polyethylene films are used in relatively thin sections compared with other liners because they are too stiff and boardy in heavier thicknesses. These thin sections do not weather nearly as well as the thicker competitive materials.

[2] Approximate; considerable variation from these costs is possible.

A NEW METHOD OF INSTALLING PLASTIC MEMBRANES[1]

C. Brent Cluff[2]

INTRODUCTION

Plastic membranes are being used as reservoir liners in increasing quantity because of their economy, particularly in areas where foundation conditions are not adapted to more rigid types of linings and where seepage control is the primary objective.

Because of their susceptibility to deterioration from ultra-violet radiation, as well as damage by wind, livestock trampling, vandalism, and so forth, plastic membranes, generally, are now protected with an earth cover. Although some reservoirs have been successfully lined with plastic membranes without using an earth cover, such reservoirs have special requirements. These requirements include the capability of immediately filling the reservoir following installation, maintaining a water pool sufficient in depth to protect the otherwise exposed plastic, and covering the plastic on bank areas or using other more weather-resistant materials such as butyl rubber. Also, effective methods of seaming must be used that not only will be watertight but also will resist wind damage until filling can take place. Failure to fulfill any of these requirements will usually lead to complete failure of the liner (*2*).

Where a plastic liner is covered immediately with soil, seaming requirements are not as stringent. In construction of a one-third acre pond at the Water Resources Research Center in Tucson in 1962, the effect of an earth cover on seepage rate was studied. A liner was made from 20-foot-wide rolls of 8-mil clear polyethylene. It was seamed in the field with a hand-clamp heat sealer, which we now know is not the best method to use. As the pond was filling, the seams were checked visually through the clear plastic liner for holes. Several holes along the seams were located by the color change of the wetted subgrade below the plastic. These holes caused a 0.23 in./day initial seepage loss. The plastic was then covered with approximately 2 inches of river silt, which reduced the seepage rate to less than 0.01 in./day. Thus, an earth cover in this example reduced seepage losses to an insignificant level. This finding is not startling, but it does support the claims by some authorities that some breaks in the plastic barrier are not too significant as long as an adequate soil cover is used. (*3*).

Considerable experience has been gained in the last 10 years in the installation of plastic membranes. The generally accepted criteria have been outlined in various papers (*2, 5, 8, 9*).[3] They are summarized below.

Site Preparation

1. Use side slopes of at least 3:1.
2. Remove objects such as sharp rocks and roots by raking.
3. Roll subgrade, if necessary, as an added precaution.
4. Use cushion layer of fine material if the subgrade is too rocky.

Placing Membranes

1. Membranes should be spread without stretching.
2. For polyethylene film use 10 percent slack.

Cover Materials

1. Clayey gravel will give best results.
2. Noncohesive materials will require greater thicknesses.
3. Use at least 9 inches of cover in canals and reservoirs subject to livestock damage.

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[1] Contribution from Water Resources Research Center, Univ. of Arizona.

[2] Asst. hydrologist, Water Resources Research Center, Univ. of Arizona, Tucson, Ariz.

[3] Quackenbush, T. H. Installation of flexible membrane canal linings. 1962. Paper presented at December 1966 winter meeting, Amer. Soc. Agr. Engin., Chicago, Ill.

4. When not subjected to livestock damage, 6 inches of cover for canals and side slopes of reservoirs should be used; bottom areas of reservoirs would require less cover.
5. At least 2 inches of cover should be used in any event to assure a successful seal.
6. Cover materials should be placed with care to avoid damage to the membrane.

CONVENTIONAL METHODS FOR INSTALLATION OF PROTECTIVE COVER FOR PLASTIC MEMBRANES

Although the above criteria have been discussed in the literature and it is generally accepted that a protective earth cover is needed, relatively little detailed information has been published regarding methods of covering plastic liners in reservoirs. More has been published on covering canal liners, but many of the techniques used on canals are not directly transferable to reservoirs. Consequently, very often the local consulting engineer is left to his own imagination or to word-of-mouth information concerning the best system to use for covering plastic in reservoirs. Quite often, to be sure that the lining will not be damaged in the covering process, the engineer may require a thicker or more puncture-resistant plastic to be used, whereas, if proper seaming and covering techniques were used, a less expensive plastic might be adequate.

A detailed report on the various methods of covering plastic is beyond the scope of this paper. However, a brief summary of some of the methods used is given as background information.

A technique used on small canals and on slopes of reservoirs involves the use of a grader to distribute cover material on the plastic by blading the cover material over the edge (*6*). In both large canals and reservoirs, a crane equipped with a concrete bucket or a dragline has been used to lift cover material onto the plastic (*1*, 7).

On bottom areas in reservoirs where 20- or 30-mil polyvinyl chloride (PVC) plastic is used, dump trucks run directly on the exposed plastic (H) to dump the cover material. Where lighter plastic is used, dump trucks place cover material on the leading edge of the plastic strip, and the cover material is then usually spread by hand (7). On some jobs, dump trucks spread cover as they are driven backwards over the exposed plastic. If the distribution from the dump truck is continuous, the truck itself does not drive on the exposed plastic.[4]

On other installations, dozers push cover material from the bank onto the plastic. In an installation near Denver, a dozer was used to make a ramp and turnaround at the bottom of the reservoir. Additional dirt was then brought in with a carryall and spread with a grader with an extended blade.[4]

Elevating graders are used for covering film; they do provide an effective way of lifting earth from the bottom of a reservoir ahead of the plastic film and conveying it onto the liner.[4]

NEW METHOD OF INSTALLATION OF PLASTIC LINERS AND PROTECTIVE COVER

Since many different methods have been tried in the installation and covering of plastic, one hesitates to use the word "new" in connection with a process. The so-called new method that I am referring to in this paper is to lay down a strip of plastic and simultaneously cover it with earth. This can be accomplished in different ways. The method we tested consists of using a plastic-laying chute that fits on the back of a dump truck, as shown in figure 1. As the truck moves forward, plastic is dispensed due to the slight tension caused by the lower roller, which also partly supports the weight of the chute. The lower roller holds the plastic on the ground until the material from the truck covers the plastic. This roller is covered with rubber to reduce the chance of damaging the film.

The first plastic-dispensing chute constructed by the Water Resources Research Center at the University of Arizona was rather hurriedly designed and built in the

[4]Personal communication with C. E. Staff.

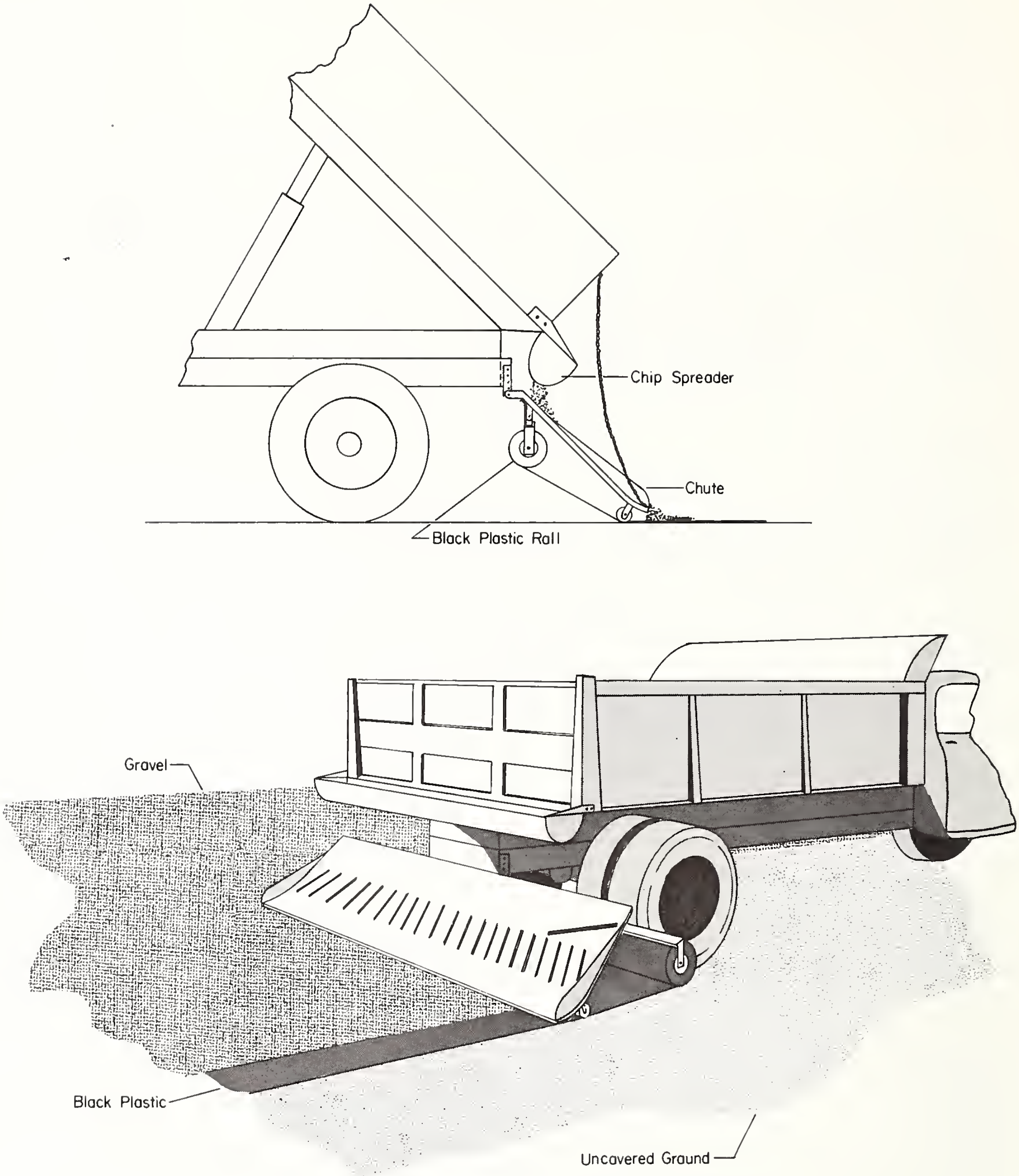

Figure 1.—Plastic-laying chute with chip spreader.

spring of 1966, for installation of a ½-acre, gravel-covered plastic catchment for harvesting precipitation (*4*). The installation of this catchment was made using funds from the Office of Water Resources Research, U.S. Department of the Interior. A. R. Hurley of the Water Resources Research Center staff was codesigner of the chute. Four-mil polyethylene plastic was successfully laid with the chute and covered with 1½ inches

of pea gravel. The plastic was overlapped on the slope, providing a continuous waterproof cover. The overlap was created by shunting gravel away from the edge as shown in figure 1.

While the dispenser was being used during the initial application, ways in which the method could be improved were noted. One improvement needed was to attach the chute to the body frame of the truck rather than the gravel spreader, so that the angle of the chute did not vary as the truck bed was elevated. This and other minor improvements were made before the chute was used again.

In February 1968, the plastic-dispensing chute was used to lay and cover a 10-mil PVC lining in a 30,000-square-foot pond at Fort Lowell Park in Tucson. This project was carried out in cooperation with the Pima County Parks and Recreation Department.

The pond was created to serve as a recreation lake and storage facility for the waste water from the park swimming pool. An attempt was made to fill the pond without the use of lining, but the seepage rate proved to be excessive, namely several feet per day as observed by park officials, since the deeper portion of the pond was excavated in sand which at one time was the bed of the Rillito Creek. It was decided to line the pond with plastic, allowing 2.5 feet of unlined freeboard in order that the swimming pool could be completely emptied into the pond at any time. The excess water will infiltrate into the banks above the plastic lining. After installation of the liner, the banks above the plastic will be grassed for both aesthetic purposes and to maintain high infiltration rates.

Figure 2.—Plastic-laying chute used on pond at Fort Lowell Park, Tucson, Ariz.

Standard procedures were followed to prepare the site for use of the plastic-laying gravel chute. The maximum side slopes were set at 3:1, and the subgrade was raked and rolled. This provided essentially a rock-free surface that was compacted sufficiently to support the weight of the loaded dump truck.

Soil, classified as gravelly loam containing 19 percent clay, that was excavated from the pond was used as cover material. Sufficient cover material was processed to provide a 4-inch layer for the bottom, increasing to a minimum of 6 inches on the side slopes. The chip spreader was not used with the dispensing chute in this application because it would impede the movement of soil from the truck. The tailgate opening of the dump truck was chained to give a 4-inch depth of cover at normal dumping speeds. To avoid plugging this opening in the gate, the cover material was screened using a 0.75-inch fixed screen devised by Pima County to fit on top of the bed of a dump truck. A pneumatic vibrator was mounted on the fixed screen to speed up processing the cover material. The dump truck was filled with material passing the screen, and oversize material was wasted off the side. The cover material was then stockpiled and reloaded later at the time of installation of the liner. For the tailgate opening required for the 4-inch cover, a 1.5-inch screen size was more than adequate to avoid plugging. Use of this size screen would speed processing in most materials so that screening could be accomplished at the time the liner is installed. This would effectively eliminate the cost of processing the cover material.

The plastic-laying chute was used to place approximately 24,000 square feet of plastic in the 30,000-square-foot reservoir. This amount of material was laid and covered in

Figure 3.—Seaming plastic in pond at Fort Lowell Park, Tucson, Ariz.

8 hours. Approximately 6,000 square feet of plastic, located on the steeper side slopes, was laid and covered using the front-end loaders and hand labor. Twelve-foot-wide rolls were used with an average overlap of 6 inches, as shown in figure 2.

The width of the cover was 10 feet. A 1-foot-wide strip was left exposed until the overlap could be seamed. The method of seaming consisted of using three beads of solvent adhesive containing a filler material, dispensed from a quart-size plastic squeeze bottle, as shown in figure 3. Following seaming, cover material was pushed onto the seams from the covered plastic by hand. Around the edge of the liner, an 8-inch trench was dug by hand to anchor the plastic sheeting.

To make the cover on the slopes more resistant to foot traffic and vandalism, the upper 10 feet of the cover material was wetted, rolled, and treated with a ⅝-inch layer of asphalt. This layer was formed by spraying a tack coat on the cover that was immediately covered with ⅝-inch chips, which were then sprayed with additional asphalt. This asphalt was subsequently covered with 2 inches of pea gravel. The completed pond is shown in figure 4.

Water-level measurements from March 1 to March 21, 1968, indicate that the rate of water loss in excess of evaporation has dropped from over 0.10 to less than 0.04 inch per day. The level of the pond during this time has been so high, because of rains and waste water from the pool, that in at least one area the water level had been above the edge of the plastic. Because of the high-water level, a significant amount of the loss is caused from water moving through the cover material into the banks above the plastic. Measurements will be made when the water level drops to determine the seepage loss through the plastic alone.

Figure 4.—Completed pond at Fort Lowell Park, Tucson, Ariz.

Table 1 includes the estimated cost of the entire operation of laying and covering 30,000 square feet of plastic, including hard work in trenching. Equipment rental includes the cost of renting the dump truck and tractor loaders.

TABLE 1. — Estimated costs for materials and installation of 30,000-square-foot plastic liner and 4 to 6 inches of cover

Item	Total Cost	Cost/ft.[2]	Cost/yd.[2]
	Dollars	Dollars	Dollars
Materials; PVC 10-mil black plastic in 12-foot rolls, including adhesive	1,295.00	0.0432	0.388
Installation:			
Common labor — 203 man-hours @ $2.50	507.50		
Supervisory labor — 24 man-hours @ $4.00	96.00		
Equipment rental	220.00		
Total installation	823.50	0.0274	0.247
Total costs	2,118.50	0.0706	0.635

DISCUSSION OF RESULTS

Although the installation of the plastic cover proceeded without serious difficulty, there were some problems. The problems encountered, and suggestions for correcting them are as follows:

(1) Keeping the truck moving in a straight line: Sudden changes in direction resulted in wrinkles at the seams which added to the time required in making a watertight bond. To help correct this situation, an inexpensive pointer can be attached to the steering mechanism of the truck so that a driver can see at all times which direction the wheels are turning. This same type of device is used on trucks used to paint straight lines on highways.

(2) Overtopping of the cover material from the sides of the truck and spilling on the overlap, increased time in cleaning up before making a seam. This could be corrected by using a larger truck and not overloading; the amount of overspill decreased as the crew gained more experience.

(3) Mounting the roll of plastic on the chute was awkward and time consuming. A simplified method is needed for handling the heavy rolls of plastic.

(4) Using the plastic-laying chute along the contour was difficult when side slopes were steeper than 4:1, because of the load shifting in the truck. Loading the truck on the upslope side corrected this. However, on most of the steeper slopes parallel to the direction the plastic was laid, installation was by hand labor. The plastic-laying chute could be used directly up and down a 3:1 slope without difficulty.

Using the same system with a 6-yard dump truck and two ⅔-yard loaders but with an improvement in equipment design, a six-man crew should be able to install at least an acre of lining and cover per day. With a 12-yard truck and a larger loader, the same crew should be able to complete as much as 2 acres per day.

The method given here is by no means the best that can be devised. It is only a beginning that hopefully will lead to more improved methods. One such improvement that could be used on large reservoirs is the use of a modified self-propelled chip spreader, with which up to a 17-foot width of plastic could be laid down and covered in one operation.

SUMMARY

Plastic liners can be successfully installed and covered in one operation. The use of the plastic-laying chute on small ponds as described in this paper, or larger self-propelled plastic-laying spreaders on reservoirs, should result in a substantial saving in time and money over the more conventional methods of installing and covering plastic. Although the amount of field seaming is increased, the effect of the inherent weakness in field seaming is minimized by the ability to immediately cover the seam. The time required to make a seam in the field is reduced because the plastic is held in place by the soil cover. The chance of damage to the film is greatly lessened since the protective earth cover is applied at the same time the plastic is laid down. This is of particular advantage with the use of plastic having a low puncture resistance, such as polyethylene.

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SOIL-CEMENT LININGS FOR WATER-CONTAINING STRUCTURES[1]

E. H. Rogers[2]

SUMMARY

Soil-cement has been used successfully as a low-cost lining material for water-containing structures for over 20 years. The more important applications include linings for domestic water supply reservoirs, farm irrigation ponds, sewage lagoons, and manmade lakes.

Soil-cement is not intended to replace concrete, which remains the material best suited for permanent linings. But, for the many structures for which a concrete lining is not economically feasible, perhaps because of a short design life or inadequate financing, a soil-cement lining can provide an economical solution.

The engineering technology of soil-cement is well understood today as a result of extensive research and wide application in road construction extending over a period of more than 30 years. Physical and chemical properties of soil-cement are predictable and controllable with the application of well-defined engineering techniques.

A wide variety of construction equipment is available to mix and place soil-cement for both small and large projects. Procedures for soil-cement construction are uncomplicated and as easily adapted to the construction of lining for a small farm reservoir as to that for a multimillion-gallon-capacity reservoir.

INTRODUCTION

Linings are installed in water-containing structures for one or more of the following reasons: Conserve water; lower maintenance costs; reclaim seeped-out areas; improve water quality; prevent weed growth; stabilize channels; and increase hydraulic capacity of channels. The ideal lining material is one that is durable, impermeable to water, structurally strong, erosion resistant, resistant to weed penetration and growth, and smooth surfaced. Additionally, the lining material should be economical. Economical here means that the total annual cost of the lining, initial cost plus maintenance cost, will be more than repaid to the owner each year by the reduction of maintenance, savings in water, and other savings in costs as a result of lining the structure. Portland Cement concrete has long been recognized as approaching most closely the ideal lining material. It has earned this reputation as a result of successful experience extending over many years. As a result, concrete has become the standard against which all other lining materials are compared.

Concrete is the most economical material for permanent linings on structures with a long design life, perhaps 50 years and more. Because of its inherently long service life and low maintenance, the annual cost of the lining is low. If the design life of a structure is short, then another lining material with a lower first cost might prove more economical on an annual cost basis.

There are many water-containing structures for which a concrete lining cannot be justified for economic reasons. It may be that the design life of the structure is relatively short, or perhaps project funds are inadequate to finance the first cost of concrete. A need thus exists for a lining material that provides the benefits of a concrete lining and yet can be built at a considerably lower first cost. To fill this need, research has been carried out over the past 25 years. The U.S. Bureau of Reclamation instituted a research program following World War II to study various lower cost canal linings (7). The Agricultural

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[1] Contribution from Portland Cement Association, San Francisco, Calif.

[2] Water resources engineer, Western Region, Portland Cement Association.

Research Service has also conducted similar research. Soil-cement is one of the materials that has been investigated as a lower cost lining material.

A close relative of Portland Cement concrete, soil-cement offers many of the advantages of concrete as a lining material but at a much lower first cost. Soil-cement, like concrete, can be made durable, practically impermeable, and erosion resistant. It also has structural properties of compressive, tensile, and flexural strength. The surface of soil-cement has hydraulic-flow characteristics similar to concrete.

Soil-cement is exactly what its name implies, a mixture of soil, cement, and water that hardens into a concrete-like material.

Soil-cement can be constructed in two distinct ways, and so it is further identified according to the method of construction as plastic or compacted soil-cement. Plastic soil-cement is made in the same manner as Portland Cement concrete and placed with the same tools and techniques. It differs only in the use of natural soils rather than selected aggregates. The cement content is in or slightly below the lower range of that for concrete. Unless concrete aggregates are costly, plastic soil-cement offers little advantage in cost over concrete.

Compacted soil-cement is made with the same techniques and equipment used in the construction of soil structures plus the added step of mixing cement into the soil. The moisture content of the soil-cement mixture is brought up to or near the optimum point to obtain a specified density. The cement content of compacted soil-cement is usually, but not always, well below that for concrete. The low cost of compacted soil-cement is the result of a combination of low-cement content coupled with low-unit placement cost of the soil-cement mixture typical of soils construction.

The origin of soil-cement is somewhat hazy, but one story dates the first use to 1915, when it was used to pave a city street. Beginning in the mid-1930's, it became the subject for exhaustive study and research as a road-paving material. World War II saw the first major use of soil-cement as a paving material for military airfields. With the advent of the great highway construction program following the war, soil-cement found ever-increasing use. Today there are thousands of miles of soil-cement roads throughout the United States.

EXAMPLES OF SOIL-CEMENT LINING

As a result of knowledge gained about the engineering properties of soil-cement from both research and practical application in road construction, engineers sought out other practical applications. Since soil-cement resembles concrete in many ways, it was a short step to experimenting with soil-cement as a substitute for concrete as a lining material for water-containing structures.

In 1945, soil-cement was first used to line the bottom of a 12-million-gallon water storage reservoir located in Port Isabel, Texas (fig. 1). The reservoir had been constructed initially with concrete-paved side slopes and no lining on the bottom. Leakage was so excessive, however, that steel petroleum storage tanks nearby were floated.

Standard ASTM soil-cement tests plus weight-loss criteria of the Portland Cement Association were used to establish the minimum cement content for a durable soil-cement at 12 percent by volume (*3, 5*). A moisture content of 18 percent gave compacted densities of 105.8 per cubic foot. Mixed-in-place procedures (*4*) were used when lining the bottom of the reservoir. A uniform layer of cement was spread on the bottom and mixed into the soil, which was then thoroughly compacted to a nominal thickness of 4 inches.

Test holes were then made around the reservoir and checked periodically for leakage. At the end of 2 years, there was no apparent leakage. After 23 years of service, the owner reports that the soil-cement is still performing satisfactorily and that maintenance has been negligible.

In 1951, the Corps of Engineers used soil-cement to line the bottom of an 8-acre reservoir near Georgetown in Washington, D.C. Six inches of the bottom soils, which varied from sandy and clayey loams to clay, were mixed with 13 percent cement. The

Figure 1.—Construction of first soil-cement reservoir lining, Port Isabel, Tex., 1945.

bottom was covered with 4,000 sq. yds. of soil-cement using mix-in-place methods; the work was completed in 5 days. When the reservoir was cleaned 4 years later, the soil-cement was found to be in good condition. The primary purpose of the lining was to prevent weed growth and facilitate cleaning. The soil-cement lining has ample structural strength to carry the heavy cleaning equipment.

Soil-cement was first used as a lining for a farm reservoir in the Coachella Valley of Southern California in 1955 (fig. 2) In the intervening years, about 12 reservoirs have been lined; four in the past year. A typical farm reservoir has a capacity of 4 acre-feet and a depth of 6 feet. Valley soils range generally from coarse sands to sandy loams. From 10- to 16-percent cement by volume must be used to make a durable soil-cement.

Farm labor has been used to build several farm reservoirs because of the simplicity of construction and minimal amount of special equipment needed. A typical sequence of soil-cement lining construction of one of these reservoirs begins with spotting bags of cement at predetermined intervals on the bottom and sides of the reservoir. The bags are broken and the cement distributed by hand-raking or by mechanical cement spreaders to a uniform depth. While the cement is being mixed into the soil with a rotary tiller, water is being added simultaneously to the mixture from a tank truck or hoses. After the cement and soil are thoroughly mixed and the moisture content judged proper by the "hand-squeeze" method (*4*), the mixture is compacted by rubber-tired road compacters or heavily loaded trucks to approximately 4 inches thick. The soil-cement is usually cured by intermittent sprinkling for several days.

Quantitative data on the amount of water losses from the farm reservoirs are quite meager. Measurements made on three reservoirs varied from one-half to one-inch drop in water level per day, or in percentage of total volume lost, 0.6 to 1.2 percent per day.

Figure 2.—Lining farm reservoir with 4 inches of soil-cement in Coachella Valley near LaQuinta, Calif., 1956. Cross-shaft mixer shown mixing cement with in-place soils.

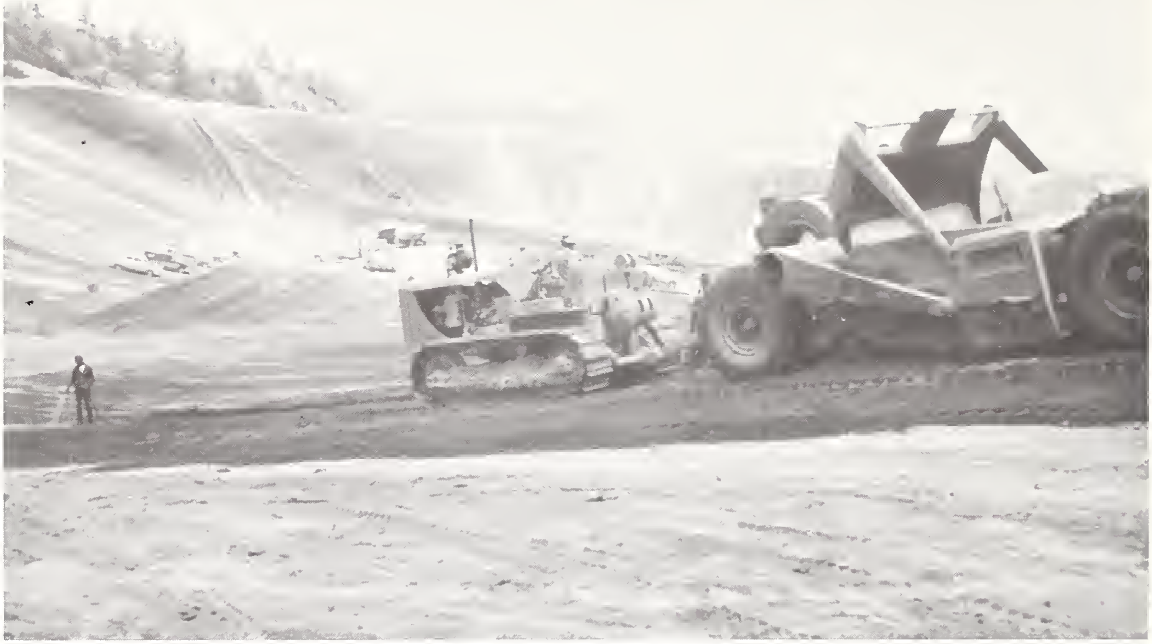
Costs of constructing a farm reservoir lining with soil-cement have changed very little in the past 13 years. In 1955, the cost per unit was about 8 to 9 cents per square foot. Today, the contract price averages 10 cents per square foot. The cost to the farm owner who does the work with farm labor is estimated at 10 to 20 percent less than the contract price.

Three larger reservoirs lined with soil-cement were constructed for the U.S. Air Force Academy near Colorado Springs in the spring of 1957 (figs. 3A and 3B). The reservoirs store water for irrigating the grounds of the Academy. Reservoir capacities are 32, 36, and 72 million gallons. The soil-cement lining is 6 inches thick, and totals 250,000 square yards for the three reservoirs.

Soil used for the lining was Dawson sand, a clean, well-graded, nonplastic material with about 8 percent passing a No. 200 sieve. The cement factor was 5 percent by weight.

The soil-cement lining was constructed using the central mixing plant method. A mixing plant was set up in the bottom of each reservoir to process soil excavated there. Scrapers were used to transport the soil-cement and spread it on the sides and bottom of each reservoir. The soil-cement was shaped and leveled with a motor grader and compacted with a caterpiller tractor. Because of the steepness of the 3:1 slope of the reservoir sides, the tractor, scraper, and grader were operated up and down the sides. Final compaction was obtained with a pneumatic-tired roller operated transversely to the side slope and assisted by a tractor. A curing membrane of MC2 oil was applied at the rate 0.2 gal. per square yard.

Six months after the reservoirs were completed, tests were made to determine the amount of water loss from seepage. Reported water losses averaged less than 1 percent per day.



A.—Soil-cement being transported and spread with a scraper. Central plant located in bottom of reservoir.

Figure 3.—72 M.G. soil-cement lined reservoir, U.S. Air Force Academy, Colorado Springs, Colo., 1957.

The most extensive soil-cement reservoir lining constructed to date was that for the Lubbock, Tex., regulating reservoir, a part of the Canadian River Project. Completed in 1966, the 500-acre-foot-capacity reservoir was designed by the U.S. Bureau of Reclamation.

The lining on the bottom is 6 inches thick. Soil-cement on the sloping sides of the reservoir is 2 feet thick to provide protection against wave action.

Primary purposes of the soil-cement lining are to provide slope protection, facilitate cleaning, and prevent weed growth.

A silty sandy borrow soil was used for the soil-cement. Cement content of slope protection was specified 12 percent by weight, and for the bottom lining 7 percent by weight (*8*). Less cement was specified for the bottom lining because it would be continuously under water and not subject to the destructive forces of weather and wave action.

Soil-cement was mixed in a central plant on the project site, hauled in trucks, and spread with a Johnson spreader. A sheepsfoot roller and pneumatic-tired roller compacted the soil-cement to 6 inches (fig. 4). The slopes were constructed by placing one

Figure 4.—Equipment train used to construct soil-cement slope protection on Lubbock, Tex., regulating reservoir, 1966.

level layer of soil-cement on top of another in stairstep fashion up the 3:1 side slopes of the reservoir. The contract price of 159,000 cu. yd. of soil-cement in place was $8.48 per cubic yard. Based on the cubic-yard price, the cost of the 6-inch bottom lining was $1.41 per square yard.

The examples of soil-cement lined reservoirs given have been limited to those used for irrigation and domestic water supply. In addition, soil-cement has been used as a lining for recreational lakes, sewage lagoons, mill ponds, and irrigation canals and ditches. From an engineering standpoint, a soil-cement lining can be used for almost anything appropriate for the use of concrete. The low cost of soil-cement provides an economical lining for those projects that would otherwise not be lined.

SOME IMPORTANT CONSIDERATIONS IN THE DESIGN AND CONSTRUCTION OF SOIL-CEMENT LINED RESERVOIRS

In this part of the paper, some of the more important considerations in the design and construction of soil-cement lined reservoirs will be discussed. For those who want a more complete treatment of the subject, the literature cited in the references should be studied.

Two essential characteristics of a soil-cement lining for water-containing structures are (1) low permeability and (2) durability. Both of these qualities are equally dependent upon good design and competent construction of the soil-cement. Since soil-cement is an engineered material, engineering skills are as important in the use of concrete as in design and construction.

One of the first steps in a soil-cement lining project is the location of the source of the soil to be used. Since soils on the project site are not necessarily the most suitable, soils located within a reasonable haul distance of the project should be investigated.

A screen analysis of all soils under investigation may be all that is needed to select the most suitable one or, at least, narrow the field of choice. Sometimes blending soils is necessary to obtain a more suitable gradation. For ease of mixing and placement and a low cement content, the soil should be a well-graded sandy, gravelly material with 100 percent passing a 3-inch screen, 55 percent or more passing a No. 4 screen, and 10 to 35 percent silt and clay combined.

Soils with oversized material or excessive amount of clay balls will require screening, increasing the costs. If the soil is poorly graded or lacking in fines, the cement content will be higher, which also increases the cost. Approximate cement contents for a wide range of soils are known, and they are most useful in evaluating the soils under study (*2, p. 138; 5*). An economic analysis is made of each soil under investigation to determine which would produce the most economical soil-cement mixture in place. For example, importing soil from a distant source may be more economical when the cement content of the borrow soil is enough lower than that of the on-site soil to more than compensate for handling and hauling costs.

The design cement factor for the soil-cement mixture is established from the results of tests conducted on the soils under investigation which relate durability and permeability to cement factor. ASTM tests D599 and 560, the wet-dry and freeze-thaw tests, were developed as a measure of the durability of a soil-cement for use in paving. The results of these tests give the percentage loss of material from a test specimen with a given cement factor after subjecting it to 12 cycles of wetting and drying or freezing and thawing. A number of specimens with a broad range of cement factors are tested to provide a range of values for the percent of material lost. The test does not indicate which of the cement factors is suitable for construction. For the test to have meaning, performance of soil-cement in service must be correlated with the tests to establish a design criteria. From a study of the field performance of many soil-cement pavements, the Portland Cement Association developed criteria for determining cement requirements for soil-cement paving based on the wet-dry and freeze-thaw tests. Allowable weight losses of soil-cement specimens in the tests are given in table 1.

TABLE 1.—Allowable soil-cement losses during 12 cycles of either the wet-dry or freeze-thaw test

Soil group	Maximum loss
	percent
A-1, A-24, A-2-5, A-3	14
A-2-6, A-2-7, A-4, A-5	10
A-6, A-7	7

The standard ASTM soil-cement tests require 6 weeks to complete. To reduce both time and expense, the Portland Cement Association developed a short-cut test procedure

for sandy soils (5). This test consists of conducting a moisture-density test, making a soils-gradation test, and molding and testing specimens of a 7-day compression test. With the soil density and gradation known, the required cement factor can be picked off curves provided in the procedure manual. The purpose of the 7-day compression test is to detect those soils that do not react properly with cement and result in abnormally low strengths.

When used as a lining material, slope protection for dams, or other similar applications, soil-cement is subject to greater destructive forces than when used for paving. To provide greater durability, the Portland Cement Association recommends that for soils containing less than 50 percent combined silt and clay the cement content should be increased by 2 percentage points higher than that established by weight losses in table 1. For soils containing more than 50 percent silt and clay, the cement content should be increased by 4 percentage points.

A criterion of 6 percent has been used by the U.S. Bureau of Reclamation as the maximum permissible weight loss in the wet-dry, freeze-thaw tests for soil-cement on slope-protection projects. In selecting the design-cement factor, data from the durability tests are first plotted as curves of weight loss versus cement content, illustrated by figure 5.

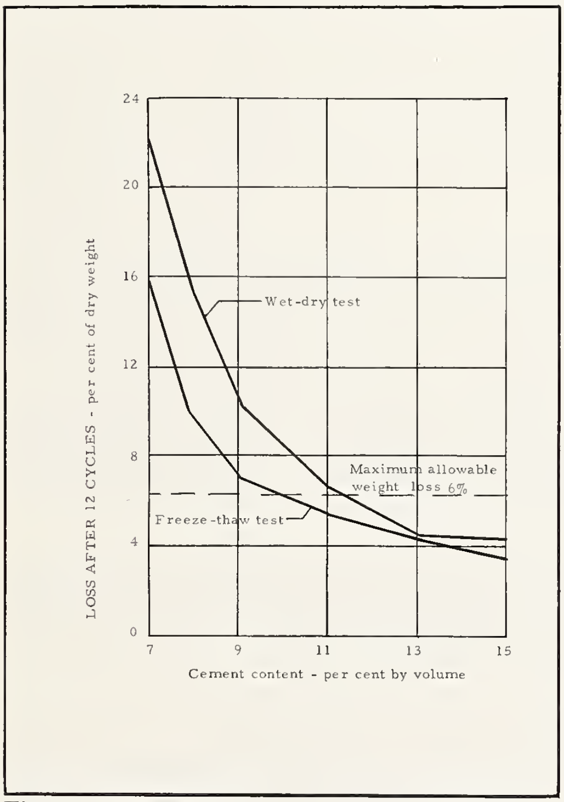


Figure 5.—Freeze-thaw, wet-dry durability test data.

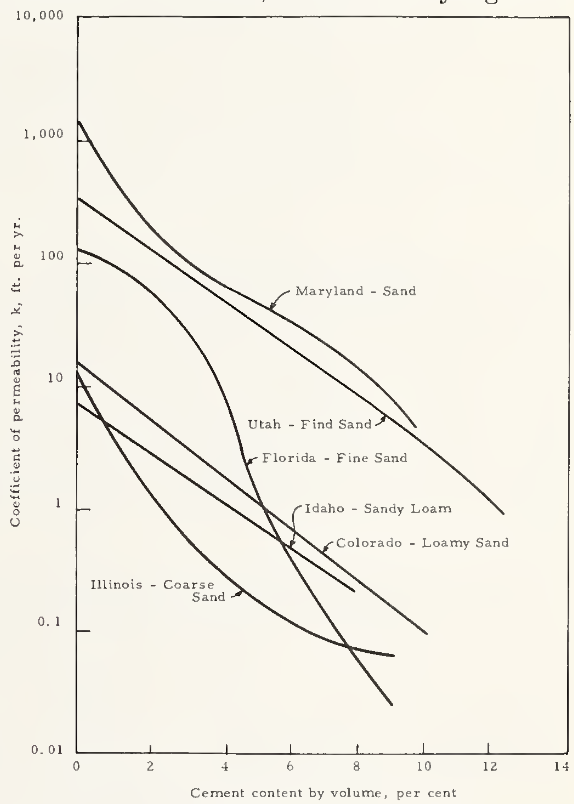


Figure 6.—Permeability of cement-treated soils.

If the slope of the durability curves is relatively steep at the 6 percent weight loss intercept, a variation in the cement content results in a disproportionate variation in weight loss. In the example of figure 5, a cement content of 11½ percent would meet the criterion of 6 percent weight loss, but a cement content of 13 percent was selected to provide a factor of safety. Thus, normal variations in the cement content of the soil-cement mixture during construction would not compromise the durability of the material. The design-cement factor would have been 11½ percent if the slope of the curves had been essentially flat as it is beyond the 13 percent factor intercept.

The U.S. Bureau of Reclamation procedure for determining the cement factor for soil-cement is somewhat more conservative than that suggested by the Portland Cement

Assocation. One reason is that soil-cement has been used for slope protection on major earth dams and large reservoirs where it must provide a long service life. For such application, the durability of the slope protection should approximate that of concrete.

A soil-cement lining that is continuously submerged is in the most favorable of environments, free from wet-dry, freeze-thaw cycles and wave action. Thus, there is no need to increase the cement factor above the amount fixed by the weight loss criteria in table 1. Considerable economy in construction of soil-cement lining may be realized by using a minimum cement factor for that part of the lining below the low-water level.

Soil-cement lining must have a low permeability if it is to be effective in conserving water. The permeability of almost all soil-cement is an inverse function of the cement content; that is, an increase in the cement factor decreases the permeability. This relationship is illustrated by figure 6 which shows curves of permeability versus cement content for a number of sandy soils. Exceptions are some very fine-grained soils that become more permeable with an increase in the cement content. Rarely would these soils be utilized for soil-cement.

There is no standardized test for determining the permeability of soil-cement. However, the equipment and procedures for measuring the permeability of concrete are adaptable. Soil-cement test specimens are prepared in the same way as they are for the wet-dry, freeze-thaw tests. A range of cement factors is used which will give permeabilities below and above the value to be used for design of the soil-cement. At least three specimens with different cement contents should be tested so a curve of permeability versus cement content can be drawn. From the curve of permeability, it is a simple matter to pick off the cement content that will produce the desired permeability for the soil-cement lining.

There are no hard and fast rules to follow in selecting the permeability coefficient of the soil-cement to be used. Soils with a permeability coefficient (k) equal to 1 foot per year or less are referred to in the literature as "impervious soils" typified by homogeneous clays below the zone of weathering. A soil-cement permeability of 1 ft./yr. can usually be obtained with a moderate cement factor, even with poorly graded soils. It is suggested as a reasonable value to adopt as a maximum allowable permeability. The design value of (k) is established only after a study of the curve of coefficient of permeability versus cement content. The shape of the curve might show that an increase in the cement content required for a $k = 1$ ft./yr. by a small percentage would reduce the permeability by one-half or more. An economic analysis is necessary to evaluate if the cost of additional cement to reduce the permeability would be justified by the value of the water saved.

There is little information concerning the permeability of soil-cement linings in service. The average permeability coefficient for the linings of the three reservoirs constructed at the U.S. Air Force Academy was 2 ft. per year or less when tested 6 months after construction. Soil used for the lining was a clean, well-graded sandy material, and the cement factor was 5 percent by weight.

Seepage measurements were made by the Soil Conservation Service on two farm reservoirs in the Coachella Valley 5 and 7 years after construction. The coefficient of permeability (k) for each reservoir approximated 1 ft. per year. Soils used for the reservoir linings were poorly graded silty sands, and the cement factor in each case was 14 percent by weight.

Studies by the Bureau of Reclamation have shown that seepage losses for soil-cement canal linings may be of the same order of magnitude as for concrete linings *(7)*.

Soil-cement can be used more efficiently and economically on level or moderately sloping ground because the equipment and construction techniques were specifically developed for road construction. Construction on an inclined surface becomes more difficult and costly with increasing steepness of the slope. Whenever feasible, the sides of a water-containing structure should be designed with a slope not steeper than 5 to 1. On such a slope, soil-cement construction is almost as efficient as it is on level ground because the construction equipment can be operated without difficulty around the sides of the structure, transversely to the slope. As the slope is made steeper than 5 to 1, the equip-

ment becomes more difficult to operate and the rate of construction drops off. Some equipment must be supported on the slopes with a cable attached to other equipment at the top of the slope or pushed with a tractor. On side slopes steeper than 3 to 1, the construction equipment must be operated up and down the side slope, a slow and costly procedure.

An economic factor that must be considered in the use of a flatter side slope is the increase in the area of lining. To illustrate, for a 2-acre-ft-capacity reservoir with a depth of 10 ft. and a 2-ft. freeboard, the area of lining is increased by two-thirds when the side slope is decreased from 3 to 1 to 6 to 1. Despite the greater area, the overall cost of the lining will generally be little different since the cost of construction on the flatter slope is substantially less. A strong argument for using a side slope between 5 and 6 to 1 is that construction operations, mixing, spreading, and compacting are more efficient, and consequently, there are less problems with controlling the quality of the soil-cement.

When steep side slopes cannot be avoided, procedures for central plant construction for compacted soil-cement should be used. Concrete, plastic soil-cement, or shotcrete are possible alternatives. Although the costs of these materials are considerably higher than compacted soil-cement, the lesser area of lining on the steeper slope is a compensating factor in the overall cost.

For a soil-cement lining to have the durability and low permeability for which it was designed, close engineering control must be exercised over the construction. Every effort should be made to obtain both the cement content and density specified for the soil-cement lining because a reduction of either will compromise both durability and watertightness. A 10-percent reduction in the cement content can double the permeability of some soils. With a 10-percent reduction in cement content, the loss in durability can be as high as 50 percent as measured by the weight loss of specimens in ASTM wet-dry, freeze-thaw tests *(2, p. 138)*.

A reduction in density also causes a loss of both durability and permeability of soil-cement. A 10-percent reduction in the standard density of some soils will reduce the durability by one-half or more as measured by weight loss in the wet-dry, freeze-thaw tests *(2, p. 138)*.

These examples are cited to emphasize the importance of proper construction to obtain quality soil-cement. It is not difficult to obtain both the proper cement content and density when proper equipment and competent workmen are employed, coupled with adequate construction inspection and materials control. Outlined below are construction-control procedures that have resulted in the construction of high-quality soil-cement.

Materials control involves primarily periodic checks on the gradation of the soil to detect variation from the limits imposed by the specification. Of particular concern is a significant change in the percentage of fines (passing #200) since the physical properties of the soil-cement are strongly influenced by the amount of fines.

The soil and cement are proportioned and mixed by either the mix-in-place or central mixing plant methods. With mix-in-place construction, if a cement spreader is used, the accuracy of the spread is checked by periodic weighing of samples of cement collected on a canvas sheet placed ahead of the spreader. This test is particularly necessary at the beginning of construction to establish the accuracy and consistency of the spreading equipment. When bagged cement is used, a visual check should be made to verify that the specified number of bags have been placed at the proper interval, and that the cement has been spread to a uniform depth.

To control mixing operations, test holes are dug directly behind the traveling mixer to check for proper depth and uniform mixing of the soil and cement. The loose depth of mixed material should be such that when compacted the soil-cement will be the specified thickness. The soil and cement are uniformly mixed when the color of the soil-cement mixture is uniform from top to bottom of the test hole. Samples can be tested in a laboratory to measure the cement content chemically. Test samples should be taken from top, middle, and bottom of the soil-cement layer to ascertain the uniformity of mixing.

After all cement has been mixed into the underlying soil, water is added to bring the soil to optimum moisture for compaction. If soil is overly dry, it can be presoaked the day

before construction. Perhaps the most useful test for control of the moisture content is the hand-squeeze method when used by an experienced person. A device for determining rapid moisture can be used for determining quantitative moisture as the soil moisture is brought to optimum.

Proportioning and mixing soil-cement with a central plant is generally more accurate and easier to control than mix-in-place construction. One governmental agency requires that the proportioning equipment must be sensitive to a 2-percent variation in the specified weight. When the plant is first placed in operation, proportioning equipment for soil, cement, and water must be calibrated within the tolerances specified over the full range of plant-operating speeds. Checking actual batch weights is necessary. Before any soil-cement is placed, enough test batches should be run to check for accuracy and to insure a smooth, uninterrupted operation. Tests for cement and moisture contents should be conducted as often as necessary to check the accuracy of the proportioning equipment.

The importance of moisture control requires that sufficient tests be made of the moisture content of the soil entering the plant so the water feed at the mixer can be adjusted. The soil-cement mixture should be deposited on the grade at a moisture content within plus or minus 1 percent of optimum. If there is a significant loss of moisture in hauling, the material should be covered with a tarpaulin.

Soil-cement from a central plant should be deposited on the grade within 30 minutes after mixing. The material should be spread in a uniform layer with equipment that can be adjusted to provide the exact depth of material required.

Soil-cement should be compacted promptly. The Bureau of Reclamation requires that compaction be completed within 1 hour after the soil-cement is spread. The moisture content of the material must be at or very near optimum when compaction begins. Compaction must be sufficient to obtain specified density. At the start of the project, in-place density tests are made of compacted soil-cement. From these tests the required number of passes of each type of compaction equipment can be ascertained. Thereafter, during construction, some minimum number of density tests are made daily. One agency makes one test for every 500 cu. yd. of soil-cement with a daily minimum of two.

After the soil-cement mixture has been compacted to the minimum specified density, curing should start immediately. Proper curing is just as important with soil-cement as it is with concrete. Improper or no curing can result in excessive shrinkage cracking, reduced durability, increased permeability, and lower strength. A continuous water cure is the most effective, followed by curing membranes such as proprietary pigmented compounds, bituminous emulsions, or cut-back oils. The structure should be filled with water as soon as practicable. If the soil-cement can be kept at or near saturation, change in volume is minimized. Shrinkage cracks also will be largely eliminated or, if formed, will be tightly closed hairline cracks.

In common with all other construction materials, the quality of completed soil-cement depends on both design and construction. If the procedures for the design and construction of soil-cement linings are followed, the maximum service performance inherent in the material will be achieved.

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CONTROL OF SEEPAGE IN EARTH DAMS

By Harry R. Cedergren[1]

INTRODUCTION

Every earth dam required to store water, even temporarily, must be capable of doing its job *safely*. Reservoirs that store water only intermittently for later safe release may appear less critical than those that must hold water permanently, such as storage and regulating reservoirs for power and domestic use. In some cases, this may be true, but a reservoir that holds water only at infrequent intervals is put to the test only intermittently; hence the owner must be satisfied that his dam is always safe, even though he may be able to observe it only when empty and dry for the greatest part of the time. Hence, flood-control reservoirs must be safeguarded against the various mechanisms that, if permitted to work uncontrolled, can lower their safety. Some conditions that must be guarded against are hidden channels formd by burrowing animals or decaying roots, the opening of cracks because of uneven settlement or desiccation (particularly in arid climates), hidden joints in erodible rocks, and the like.

Many of the failures of earth dams *(6,* pp. 116-123) have been caused by piping along outlet pipes, beside or under spillway walls and slabs, or along other contacts between natural or compacted earth and rigid structural members. If the designers and builders of dams give sufficient attention to fundamental principles of seepage analysis and control, and the work is carried out under carefully prepared plans and specifications, seepage problems can be virtually eliminated. As in all other civil engineering works involving soil mechanics and foundation problems, a high level of safety for earth dams requires the following important steps:

1. Thorough field investigations to reasonably establish soil and geological conditions at project sites.

2. Careful, experienced interpretations of field conditions.

3. Adequate studies and analyses based on physical properties of important earth materials.

4. Development of safe, economic designs.

5. Adequate specifications and construction carried out in accordance with these specifications.

6. Adequate instrumentation and observation of finished works.

This paper is concerned primarily with methods for analyzing seepage and developing methods for its control (primarily steps 3 and 4).

The major causes of trouble from seepage are of two types:

a) Those caused by the migration of soil particles or soft, erodible rock particles to unprotected seepage exits.

b) Those caused by lack of control over saturation. If soil or soft-rock particles are ever permitted to start washing out through unprotected exits, complete failure takes place unless the process stabilizes. In addition, if the spread of saturation and the development of seepage forces and uplift pressures are not adequately controlled, severe sloughing, heaving, or other damaging actions may take place.

A high level of protection against the damaging actions of seepage usually can be provided by two basic methods, used alone or in various combinations: (1) Those that *keep the water out* of places where it can cause damage, or *reduce the seepage rate;* and (2)

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those that control the seepage by *drainage methods* that safely remove the water which enters. Many methods can be used. This paper discusses only a few basic methods.

CONTROL BY REDUCING SEEPAGE QUANTITY

Seepage through and under earth dams has been controlled by numerous seepage-reducing methods such as open cutoffs backfilled with compacted impervious earth, asphalt-plank facings, butyl and other thin linings, grouting of foundations, slurry trenches, and sheet-pile walls. All thin cutoffs and blankets depend upon a high level of perfection; and as demonstrated by Casagrande *(2)*, minor flaws in thin cutoffs can seriously reduce their effectiveness in controlling seepage. Casagrande is a strong advocate of the use of drainage methods for controlling seepage through and under dams.

While the various special impervious membranes have been used successfully as aids in controlling seepage, earth dams are usually more economical if suitable materials are available within reasonable haul distances from proposed damsites. A common method used in controlling seepage through earth dams, particularly smaller structures, is by the construction of "zoned" dams. Some fundamental factors underlying the control of seepage in dams will be illustrated with reference to a typical zoned earth dam. For zoned dams to be practical, liberal supplies of two distinctly different classes of materials are needed, such as (1) clay, or other relatively impermeable soils or highly weathered rocks, and (2) sandy gravel, or other strong, permeable materials. Further, if zoning is to be highly effective, the foundations and abutments must be relatively watertight.

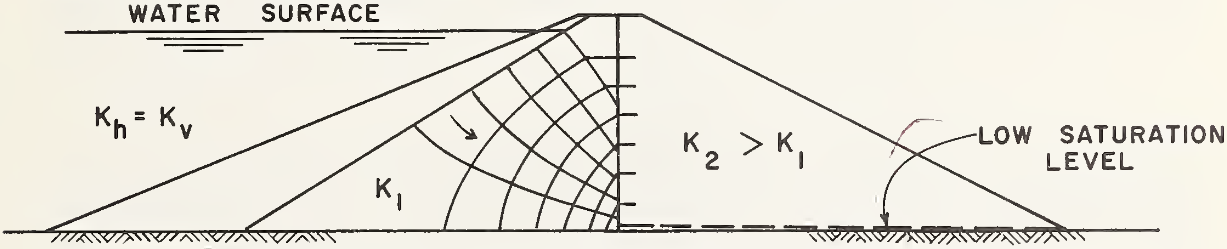

Figure 1.—An ideal zoned-earth dam on an impervious foundation. Note low-saturation level in dam.

An ideal zoned earth dam is shown in figure 1. Here, leakage through the dam and its foundation is very small, and the downstream "shell" of sandy gravel or other relatively pervious materials available near the damsite is hundreds of times more permeable than the upstream "impervious" zone. As shown, the saturation level in the downstream part of the dam is very low. Since well-drained earth is heavy and relatively strong, the dam in figure 1 has a high level of safety against failure of the downstream slope, both under normal-operating conditions and during severe earthquakes.

If the dam in figure 1 is constructed on a relatively permeable earth or jointed rock

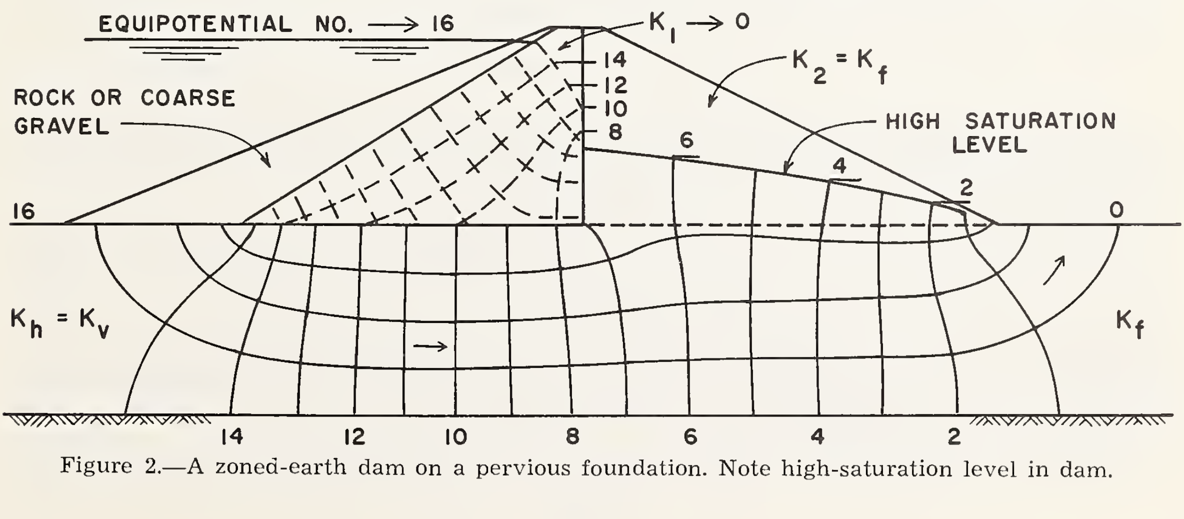

Figure 2.—A zoned-earth dam on a pervious foundation. Note high-saturation level in dam.

formation, the downstream shell must be substantially more permeable than the foundation, if it is to adequately control seepage. If it is not, seepage will rise high in the dam, as in figure 2, greatly lowering its stability. The dam in figure 2 is assumed to be identical in every way to the one in figure 1, except that the foundation is many times more pervious than the upstream "impervious" core, and the foundation is assumed to have the same permeability as the downstream pervious section of the dam. The flow net in figure 2 was drawn with the assumption that the core is extremely impervious and contributes negligible seepage. The seepage rises to a high level in the downstream part of the dam, showing that zoning alone may be ineffective in controlling seepage through dams on pervious foundations.

When foundations are relatively permeable in relation to the impervious sections of dams, seepage often can be reduced by the use of compacted upstream impervious blankets, or by various impervious cutoffs. The potential benefits of cutoffs for controlling underseepage are illustrated by the flow net solution in figure 3. In developing this flow net, it was assumed that by grouting, or other means the permeability of a portion of the

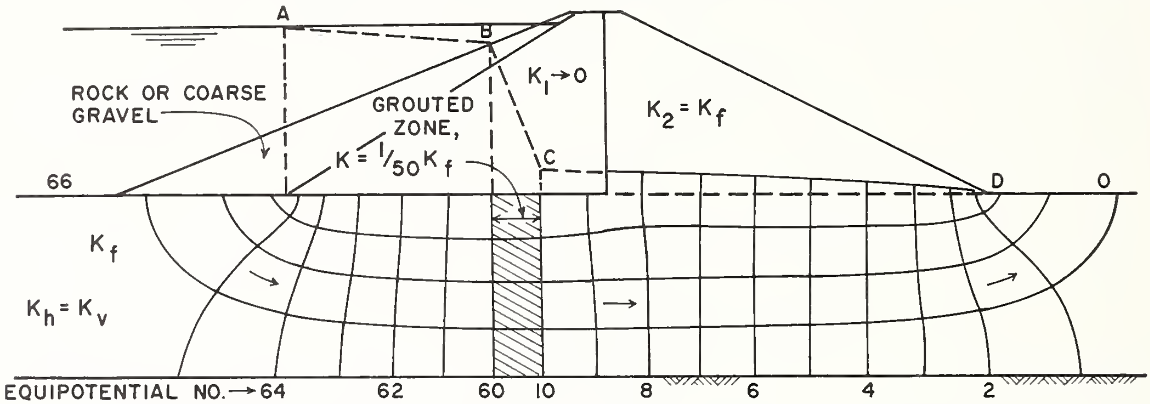

Figure 3.—Same dam as in figure 2, but with grouted zone in foundation.

foundation (the shaded zone) has been lowered to 1/50th of that of the natural foundation. For clarity in illustrating the method of analysis, the treated zone is assumed to be relatively wide; however, the same principles could be applied to narrower zones. The head losses at the top of the foundation are shown by line *A-B-C-D* (fig. 3), which was obtained from the flow net, recognizing that the treated zone consumes 50 times as much head as an equal width of untreated foundation. Thus, the number of equipotential drops in the untreated part of the foundation is 16, and the total number of equipotential drops in the foundation of the dam is 50 + 16, or 66; and 50/66 of the differential head h is used up in the treated zone. By this method the potential benefits of various kinds of treatments can be analyzed. The flow net is a handy working tool for making such studies.

CONTROL BY DRAINAGE

Numerous kinds of seepage-reducing methods have been used for controlling seepage through and under earth dams. Since these methods depend on a high degree of perfection, which cannot always be obtained, they generally should be backed up by a "second line of defense" in the form of some kind of drainage.

Often, drainage is the *primary* means for controlling seepage in dams and their supporting abutments and foundations. Correctly designed and constructed deep-seated drainage offers permanent security against the damaging actions of seepage. Not only is the safety and stability increased under normal operating conditions, but resistance to earthquake shocks can be greatly increased. Well-drained, well-compacted earth masses are virtually undamaged by shocks, whereas saturated earth masses are considerably more vulnerable to damage, particularly if they are not sufficiently dense to resist volume changes under the effects of severe shaking.

The potential benefits of positive drainage are illustrated in figure 4, which shows

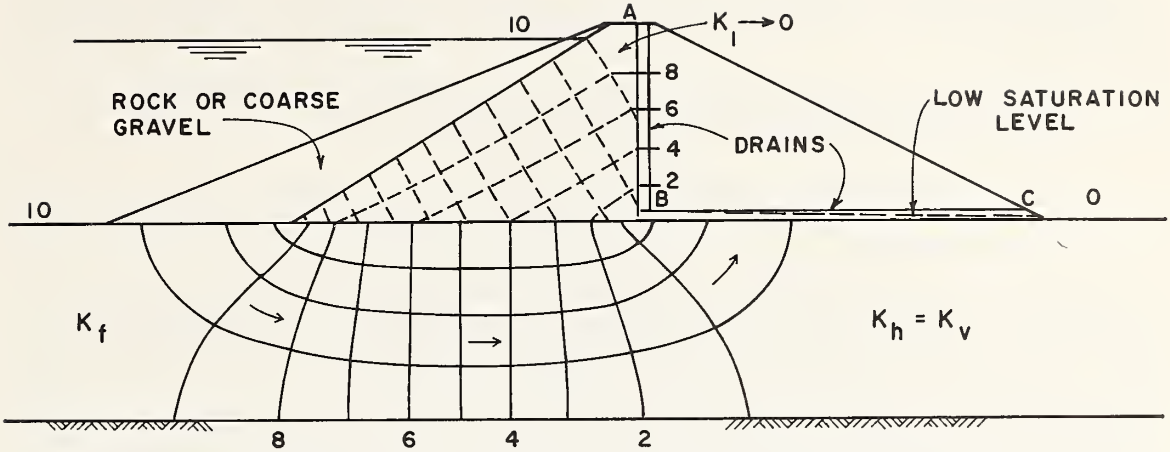

Figure 4.—Same dam as in figure 2, but with positive internal drainage.

an earth dam of the same overall dimensions as in figure 2, with the same impervious zone as in the dam in figure 2. The only changes are a positive vertical "chimney" drain from *A* to *B*, and a horizontal outlet drain from *B* to *C*. The great improvement in seepage control is readily apparent.

Piping Protection

Whenever seepage flows into a drain such as from *A* to *B* and from *B* to *C* in figure 4, or into the more pervious downstream zone in figure 2, the migration of all erodible soil or soft, weathered rocks through the more pervious material must be prevented. Security against piping at seepage interfaces, as just described, can be assured by prohibiting the use of materials containing pore spaces large enough for soil particles to pass through. Safety against piping can be assured by using the well-known "Terzaghi" filter criterion *(1; 4,* pp. 175-179; *7,* pp. 134-135; *8,* pp. 50-51):

$$\frac{D_{15} \text{ of filter}}{D_{85} \text{ of protected soil}} < 4 \text{ or } 5. \qquad (1)$$

Most major builders of earth dams *(9, 10)* require all filter and drain materials in earth dams to meet the criterion represented by equation 1, with the exception that a ratio of up to 8 or 10 frequently is permitted for plastic clays. Equation 1 states that the 15-percent sized filter shall be not more than 4 or 5 times the 85 percent size of a protected soil.

Other secondary requirements for the grain sizes of filters are also frequently imposed, but in most dams if equation 1 is satisfied in every part of a filter, a high level of security against piping is assured.

When building earth dams, levees, and reservoirs, tight control of construction is needed to avoid unsatisfactory conditions that can lead to internal erosion and eventual piping failure. Segregation that produces extensive pockets of materials that do not satisfy equation 1 is common. Segregation usually can be controlled by keeping filter materials thoroughly moistened and by avoiding the use of excessively broad ranges of sizes and too large sizes of aggregates in filters.

Discharge Capacity

If drains such as are shown in figure 4 are to do their work properly, they must not only be designed to prevent piping, but they must be capable of removing all the seepage that reaches them without large head requirements. If seepage is flowing *across* the thin dimension of a filter or transition, and into a much more permeable zone, the filter or transition need be only a few times more permeable than the soil being protected. But, if

a filter must serve as a comparatively thin *conveyor* or *conductor,* as horizontal blanket *BC* in figure 4, the drain should be designed as a hydraulic conveyor or conductor *(3; 4,* pp. 184-189; *5)*.[2]

In designing drains as hydraulic conveyors, three important steps are required:

1. Estimating all probable seepage to be removed.

2. Calculating the required *conductivity* or *transmissibility* of the drain.

3. Selecting a practical thickness and permeability of the drain that will assure adequate seepage discharge.

Seepage rates through dams and foundations usually can be estimated by drawing flow nets as in figures 1 to 4, and computing the quantity from the following equation:

$$q = kh\frac{n_f}{n_d}. \tag{2}$$

In equation 2, q is the unit seepage quantity per linear foot of a dam or other hydraulic structure, h is the differential head causing the flow, and n_f/n_d is the *shape factor* which is determined by counting the number of flow channels n_f and the number of equipotential drops n_d, and determining their ratio.

Seepage rates can also be estimated with Darcy's law:

$$q = kiA, \tag{3}$$

if the effective permeability k, the effective hydraulic gradient i, and the effective cross-sectional area of the soil A (normal to the direction of flow), can be estimated with reasonable dependability.

After the seepage rate has been estimated, a drain can then be designed to assure sufficient capacity to discharge at least several times the estimated total seepage without excessive buildup of head in the drain. Usually drains should be capable of removing at least 5 to 10 times the theoretical minimum requirements, since extraneous sources of seepage frequently overload drains if no appreciable factor of safety is provided.

A useful and practical criterion for designing drains for discharge capacity can be obtained by rearranging Darcy's law in the form:

$$kA = \frac{q}{i}. \tag{4}$$

In equation 4, kA represents the minimum product of drain permeability and thickness that will remove seepage quantity q under a hydraulic gradient i. The value of i used in equation 4 should be the maximum the designer feels can be allowed in a drain. It should always be small enough to prevent the rise of saturation into zones that should be kept well drained. Thus, in horizontal drain *BC* in figure 4, the hydraulic gradient usually should not be greater than a few percentages.

To provide a reasonable "factor of safety" in discharge capacity, the theoretical value of kA obtained with equation 4 generally should be multiplied by at least 5 or 10. This method is applied to the design of a drain for an earth dam in the following example, which refers to figure 4:

Example of Designing for Discharge Capacity

Assume that a q of 20 cu. ft./day/linear foot of the dam must be discharged horizontally to the toe at *C* by drain *BC*, and that a maximum hydraulic gradient of 0.04 can be allowed in the drain. Use equation 4 to estimate the required permeability and thickness of the drain, allowing a factor of safety of 10.

From equation 4, kA = q/i = (20 cu. ft./day)/0.04 = 500 cu. ft./day/linear foot. Using a factor of safety of 10, kA = 10(500) = 5,000 cu. ft./day/linear foot. Any practical,

[2] Cedergren, H. R. and Lovering, W. R. The economics and practicability of layered drains for roadbeds. 1968. Paper presented at the 47th annual meeting of the Highway Res. Bd., sess. 37.

economical combination of k and A having a product of at least 5,000 will be adequate. Investigate several local commercial aggregates which are assumed to be available at approximately the same price per cubic yard.

First trial: Assume washed concrete sand with a permeability of 10 ft./day. The required thickness A = kA/k = 5,000/10 = 500 ft. (This obviously is an impractical solution.)

Second trial: Assume washed uniform-sized pea gravel ¼ to ⅜in. in size with k = 4,000 ft./day. The required thickness is 5,000/4,000 = 1.25 ft.

Third trial: Assume washed one-sized gravel or ½ to ¾-in. crushed stone with k = 30,000 ft./day. The required thickness is 5,000/30,000 = 0.16 ft.

Suggested solution: Design as a *graded filter* with a core of pea gravel or other one-sized aggregate ranging in size from ¼ to ¾ in., sandwiched between protective fine filters. Check all layers for compliance with equation 1 (the fine filters must be compatible with the adjacent embankment and foundation soils, and the coarse filter must be compatible with the fine filter).

Note that the fine filters must be sufficiently permeable to allow seepage to enter the highly pervious core without excessive restriction. The quantity that can enter the drain between *B* to *C*, for example, can be verified with Darcy's law q = kiA, by multiplying the permeability of the fine filter by an allowable hydraulic gradient across the fine filter (say 100 percent), and the distance from *A* to *B*. A relatively large gradient usually can be tolerated across a fine filter, since the water is required to travel only a small distance (its thickness, which usually is not more than a few feet). For example, if the thickness of the fine filter is 2 ft., a differential head of 2 ft. would produce a hydraulic gradient of 100 percent.

SUMMARY

When water is stored behind dams in reservoirs, an energy differential is created, producing pressures and forces in the water that must be controlled to prevent loss of life and damage to property. The seeping water causes two major kinds of problems: (a) Those due to piping, and (b) those due to uncontrolled rise and spread of saturation. Protection from seepage usually can be provided by methods that either reduce the quantity of seepage, or control it by drainage. Although each of these two basic methods is sometimes used substantially alone, combinations of these methods usually are employed. The designer should always consider alternate methods that are available in relation to soils and geological conditions at individual sites. The experimental and analytical soil mechanics methods, when applied with experience and judgment, can virtually eliminate serious troubles and aid the designer in obtaining a high level of seepage control at the least overall cost.

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DISCUSSION

SESSION III — SEEPAGE CONTROL

A. McQueen: What was the type of ditch material for the high side slope application of fiberglass and asphalt lining?

L. Myers: The soil in question was a loam containing 40 percent sand, 46 percent silt, and 14 percent clay. It had not been compacted and serious problems would have developed if a low-strength lining had been used.

A. McQueen: What was the method of upslope anchoring of this material?

L. Myers: The upper edge of the lining was buried in a small trench. This was done primarily to protect the ege of the lining. The entire lining was bonded to the underlying soil with asphalt. Anchoring the upper edge of the lining to hold the lining in place, as is necessary with unbonded plastic and rubber sheeting, was not required.

R. Worstell: Is the asphalt-fiberglass lining especially adaptable to lining existing canals and ditches with side slopes exceeding the 3 to 1 ratio required for other plastic and rubber linings?

L. Myers: Asphalt-fiberglass linings are well adapted to lining ditches and small canals with relatively steep side slopes. Side slopes on the ditch we lined were steeper than 1 to 1. It should be mentioned that this lining is easy to shape and bond to structures in the ditches. We have no experience with the material in large canals. I believe, however, that properly installed asphalt-fiberglass can be used as an exposed lining in large canals.

A. McQueen: What were the compaction and other specifications for subsurface preparation prior to the lining installation?

B. Cluff: The subsurface was hand raked and all rocks larger than ¾ inch were removed. Following a 1-inch rainfall, the surface was rolled with a ½-ton roller. Density measurements in the subsurface were not taken. The only criteria used was that the subsurface be free of rocks and firm enough to support the weight of a loaded 6-yard dump truck. This requirement was easily attained by hand raking and rolling.

D. Shockley: What is the thickness of soil-cement linings?

E. Rogers: The two common thicknesses of soil-cement linings are 4 and 6 inches. Soil-cement construction methods have generally dictated the lining thickness. Six inches of soil-cement is about the maximum thickness that can be constructed easily in one lift. Four inches has been adopted as a practical minimum lining thickness, based on experience.

R. Worstell: Is sulfate-resistant soil-cement required in alkali areas?

E. Rogers: Yes, a sulfate-resistant cement should be used in alkali areas. The guidelines set forth for portland cement concrete should be followed.

F. Cooper: What are the mechanisms of cracking in soil-cement linings?

E. Rogers: The mechanism of cracking in soil-cement is the same as that for concrete. Drying of the soil-cement or a drop in temperature cause tensile stresses to develop which may exceed the strength of the soil-cement and cracking results. Cracks in soil-cement reservoir linings will usually be closed when the lining is in a saturated condition.

F. Cooper: Would secondary additives or reinforcements to the soil-cement improve its performance by preventing cracking?

E. Rogers: Presently there are no practical or economical materials or procedures to

prevent the cracking of soil-cement. If a soil-cement lining can be kept at or near saturation after construction, cracking will be minimized.

D. Raymond: Was the cost of 10 cents/sq. ft. experienced in larger structures, such as at the Air Force Academy and at Lubbock, Tex.?

E. Rogers: Unit cost of the U.S. Air Force Academy reservoir lining is not available. The Lubbock Reservoir lining cost 15 cents/sq. ft. Since this lining is 6 inches thick, the cost is proportional to the 4-inch-thick linings costing 10 cents/sq. ft.

L. Lawhon: What is the possibility for self-healing of cracks in soil-cement linings?

E. Rogers: I have no evidence to support the existence of autogenous healing of cracks in soil-cement as it is experienced in concrete.

S. Resnick: How did you derive the permeability curves shown in your slides?

E. Rogers: The permeability curves are derived from permeability tests performed on soil-cement specimens prepared in the laboratory.

SESSION IV. — SEEPAGE DETECTION AND EVALUATION

Chairman: Tyler N. Quackenbush

REVIEW OF METHODS FOR MEASURING AND PREDICTING SEEPAGE[1]

Herman Bouwer and Robert C. Rice[2]

INTRODUCTION

Quantitative information of seepage loses from irrigation or natural channels can be obtained by direct measurement on the channel or by calculation. The second approach, in which seepage is calculated from the hydraulic conductivity of the soil materials and the boundary conditions of the flow system, will be of particular value for canals that are still in the planning stage, for example, in determining desired canal capacities or canal locations for minimum total seepage losses. The methodology for direct measurement or calculation of seepage from channels or reservoirs has progressed sufficiently to enable acquisition of quantitative seepage data for a wide variety of conditions.

SEEPAGE MEASUREMENT

Methods available for measuring seepage rates from open channels are the inflow-outflow, ponding, seepage meter, and salt-penetration techniques (*7*, *8*, *10*).

Inflow-Outflow Technique

The inflow-outflow technique (*10*) is in principle the best method, because it yields the seepage for the entire canal section in question and under normal-operating conditions. The main disadvantage is that, unless the seepage rate is very high or the channel section is very long, seepage is evaluated as a small difference between two relatively large numbers (the inflow and outflow rates) which themselves are somewhat inaccurate. Thus, extremely careful measurement of the inflow and outflow rates is required to minimize error. Where the technique is applied to long canal sections, steady-state conditions may take a long time to be established and leakage or outflow from turnouts and other structures may have to be evaluated before the seepage rate can be measured accurately. Also, once the seepage for a long canal section is evaluated, the question still remains of how the seepage is distributed. Knowledge of what parts of a canal have the greatest seepage losses may be important in assigning priorities in canal-lining programs where limited funds enable only certain parts to be lined.

Ponding Technique

The ponding technique (*10*) can be applied to smaller sections of a canal. However, it has the disadvantages of interrupting the service of the canal (in addition to the cost of constructing the dams) and of yielding the seepage for the artificial condition of zero velocity in the channel. Although velocity in itself has no effect on seepage (*6*), the sedimentation of suspended material that would normally be carried downstream may result in severe underestimation of the normal seepage rate. This was demonstrated by a study in which a seepage meter was left in the bottom of the Beardsley Canal (north-

[1] Contribution from the Soil and Water Conservation Research Division, Agricultural Research Service, U.S. Department of Agriculture.

[2] Research hydraulic engineer and agricultural engineer, respectively, U.S. Water Conservation Laboratory, Phoenix, Ariz.

west of Phoenix) for several days. The falling-level reservoir was kept below the water surface in the canal, except when a seepage measurement was made. Inside the seepage-meter cylinder, zero horizontal velocity occurred, and suspended material could settle on the canal bottom enclosed by the meter. The water in the canal was fairly turbid (the water depth was 3 ft. and the bottom could not be seen), and the velocity was estimated as 1.5 to 2 ft./sec. The following seepage rates were measured.

	Seepage meter No. 1	Seepage meter No. 2
24 July 1961	9.6 ft./day	——
26 July 1961	6.2 ft./day	——
27 July 1961	4.6 ft./day	——
28 July 1961	3.0 ft./day	0.82 ft./day
1 Aug. 1961	1.9 ft./day	0.18 ft./day

Since the canal was in continuous operation for at least 2 months before these measurements, the seepage rate should be relatively constant to time. Therefore, the drastic decrease in the measured rates must be attributed to sedimentation inside the seepage meter. In view of this, the best way to carry out a ponding test would be to maintain a constant water surface in the ponded section, stopping the flow into the section periodically to measure the rate of fall of the water surface for a relatively small distance of fall, and extrapolating the seepage rates back to when the ponding test started.

The inflow-outflow and ponding techniques are applicable regardless of the canal or soil conditions. They can be used where rocky bottoms or heavy weed growth make the seepage meter and salt penetration techniques difficult to use.

Seepage Meter Technique

Local seepage measurements can be obtained with the seepage meter, which is essentially a covered cylindrical infiltrometer or seepage "cup" about 1 ft. in diameter (*1*, *7*, *10*). The cylindrical part is pushed a small distance into the channel bottom and seepage is measured as the outflow from the cylinder. Two requirements to be met are (1) the seepage cup must be so designed and installed to give a minimum of soil disturbance, and (2) the outflow from the cup must be measured when the water pressure inside the cup is exactly equal to that due to the free-water level in the channel (*7*). To meet the first requirement, the seepage meter cup developed at the U. S. Water Conservation Laboratory consists of a thin-walled cylinder with beveled edge and a flat, removable lid (*7*). Before the seepage meter is pushed into the bottom material, the lid is removed to avoid pressure buildups inside the cup while the cup is pushed down. This eliminates the danger of blowouts or piping in the bottom soil beneath the cylinder. The depth of penetration of the seepage meter should be as small as possible (1 in. or less).

The second requirement is met by using a U-tube manometer to register the pressure inside the seepage cup and in the "free" water adjacent to the cup. A falling-level reservoir is connected to the seepage cup and the seepage rate is calculated from the rate of fall of the water level in this reservoir (as measured with the manometer) at the instant that the pressures inside and outside the seepage cup are the same (*7*).

In channels with high-water velocities, the average pressure head around the seepage cup is somewhat below that due to the free-water level in the channel. The magnitude of this pressure difference, Δh, can be evaluated from a table in (*6*). The seepage rate is then calculated from the rate of fall in the falling-level reservoir when the pressure head inside the seepage cup is Δh below the water level in the channel. This correction may be desirable in fast-flowing canals with coarse sand or gravel bottoms. Normally, however, the velocity effect on the pressure environment of the seepage meter cup can be ignored (*6*).

If the equal-pressure condition and the minimum soil disturbance requirements are met, the seepage meter should give an accurate measure of the seepage rate for the part of the channel bottom that is enclosed by the seepage cup. Because this area is small and

because seepage and bottom conditions vary from point to point, a number of measurements will usually be required to adequately evaluate the average seepage for a certain canal section. Jensen (*9*) reports that with the technique described, 18 seepage measurements were sufficient to yield a 15 percent standard error in the mean.

In addition to seepage, the seepage meter enables evaluation of the hydraulic conductivity of the bottom material. Where the seepage is controlled by a thin, slowly permeable layer at the surface of the bottom, the seepage meter technique can be used to determine the hydraulic impedance of this layer (*7*).

The seepage-meter technique is difficult to apply in deep canals, particularly if the water is turbid or flowing at high velocity, and in canals with heavy weed growth or rocky or gravelly bottoms.

Salt-Penetration Technique

The salt-penetration technique is essentially a tracer method, whereby seepage is calculated from the rate of advance of salt in the bottom material. For this purpose, a layer of salt is maintained on the bottom of the channel for abut 10 to 20 minutes. The salt will start to dissolve and part of the salt will enter the bottom material with the seepage flow. When the salt is completely dissolved and has disappeared from the canal bottom, normal channel water will enter the bottom again. Thus, the salt concentration distribution in the bottom material will have the form of a wave.

If the salt was present on the channel bottom for a relatively short time (about 20 minutes, for example), the wave will have a definite peak. If the salt was present for longer periods (as may occur in ponds or very sluggish canals), the wave will have a flat peak or plateau. Column studies have shown that the peak of the wave or the lower point of the plateau advances essentially as piston flow for at least the first 5 inches or so. Thus, if the rate of advance of the peak or lower point of the plateau is measured, the seepage rate can be calculated by multiplying the rate of advance of the salt by the porosity of the soil (*8*).

The depth of the peak or the lower point of the plateau of the salt concentration wave is measured with an electrical conductivity probe. This probe is slowly pushed into the bottom material, until the conductivity just starts to decrease again after having reached a peak value. The depth of the probe at this point is recorded. Dividing this depth by the time since salt was applied then yields the average rate of advance of the salt wave.

The porosity of the bottom soil can be measured on core samples. This porsity, however, usually ranges from about 0.4 for dense bottom material to about 0.6 for loose material. Thus, estimating the porosity on the basis of a visual inspection of the bottom material instead of measuring it on core samples will often yield sufficiently accurate results.

The type of salt that can be used with the technique depends on the soil conditions. If the bottom material contains little or no clay, sodium chloride in the form of rock salt can be used. Where the soil contains clay, calcium chloride or aluminum sulphate are preferable to avoid dispersion of the clay fraction and resulting reduction in seepage. If high water velocities in the channel make it difficult to maintain a layer of salt for the required 20 minutes or so, the salt can be placed in a burlap sack or similar device. This sack is then allowed to stay in place for about 20 minutes before it is moved to another location. With one man on each side of the channel and a rope on each corner of the sack, the sack is easily maneuvered around.

The salt-penetration technique yields a point measurement of the seepage. However, within the area covered by the salt, which may range from 1 to several square yards, numerous point measurements can be made. A number of such areas will then be necessary to obtain an adequate evaluation of the seepage rate from a certain channel section or reservoir.

Field studies have shown that the technique is capable of yielding quite accurate evaluations of the seepage rate (*8*). The technique is relatively simple to use and can be applied to deep canals with turbid and/or fast-flowing water, and to adverse bottom conditions.

SEEPAGE PREDICTION

Steady-State Systems

A difficult aspect of calculating seepage via an analysis of the underground flow system is the proper evaluation of the soil hydraulic conductivity, K. Measurement of K should be done at different soil depths to a distance of at least five channel-bottom widths (for trapezoidal channels with 1:1 side slopes) below the channel bottom, or until much more, or much less, permeable material than that imediately below the channel is reached, whichever comes first (*2*). Bouwer (*4*) reviews the various field techniques for measuring K, either below or in the absence of a water table.

Knowing the K-values of the soil, the position of the ground-water table, and the shape and water depth of the channel, the seepage can be obtained through an analytical solution of the flow system. Exact and approximate solutions of such systems, which are usually considered as two-dimensional, are available for a variety of conditions ((*5*) and references therein). Where the soil or boundary conditions are too complex for an analytical approach, solutions can be obtained with numerical procedures (executed with digital computers), electric analogs, or viscous or physical flow models (*5*).

Dimensionless graphs, determined by resistance network analog, and relating seepage to the depth of the water table at great distance from the canal (including infinite water table depths) were presented in a previous paper (*5*). The soil below the canal was considered uniform and isotropic and the lower boundary of the flow system was taken as an impermeable, or infinitely permeable layer. The depth of the lower boundary was taken as a variable and curves for different values (including infinity) of this depth are presented in the graphs. The analyses apply to trapezoidal canals with 1:1 side slopes and graphs were prepared for three different water depths in the channel. Thus, knowing K of the soil in which the canal is located, the position of the water table, the subsoil conditions (depth of material of much higher or much lower hydraulic conductivity than that of the soil in which the canal is imbedded), and the shape and water depth of the canal, the anticipated seepage can be evaluated from the appropriate graph. Because sedimentation and other "aging" processes causing reduced seepage rates are not taken into account, the seepage rates evaluated in this way should be considered as the maximum that can be expected. Such rates may occur when the canal begins operating or after it has been cleaned.

Transient Systems

In watershed runoff studies, it will be desirable to know the seepage rate when water enters a dry channel and the decrease of the seepage rate with time. This is a problem of transient flow that can be handled as one of two-dimensional infiltration. Though accurate analytical and numerical solutions of the one-dimensional infiltration problem are available (*11*, *12*), the necessary input information regarding hydraulic properties of the soil in relation to water content and pressure is difficult to obtain. Moreover, the infiltration rate is primarily controlled by the hydraulic conductivity at the upper ranges of the soil water content. Thus, a simplified solution such as by Green and Ampt may be more appropriate ((*5*) and references therein). With Green and Ampt's approach, the flow system is treated as one of piston flow, that is, a wetted zone of constant K and a sharp wet front are assumed. This assumption is reasonably valid, particularly if the water depth in the channel is relatively large. The infiltration rate for a system of one-dimensional vertically downward water movement is then expressed with Darcy's equation as follows:

$$i = K \frac{H_w + L_w - P_w}{L_w}$$

where i = infiltration rate at point in question,

K = hydraulic conductivity of wetted zone,

H_w = water depth at point in question,

L_w = depth of wet front, and

P_w = (negative) pressure of soil water just above wet front.

Quantitative information regarding P_w and also of K, can be obtained with the air-entry permeameter (3). The rate of advance of the wet front can be described as

$$\frac{dL_w}{dt} = \frac{i}{f},$$

where f is the difference between the volumetric water content before and after wetting of the soil. The accumulated infiltration, I, can be expressed as

$$I = fL_w.$$

These equations can be applied to the entire wetted perimeter of a channel to determine the relationship between seepage rate (or accumulated seepage volume) and time after water has entered a dry channel. For a detailed description of this procedure, reference is made to (5), where, for example, figure 26 shows successive positions of the wet front below a trapezoidal channel. Also, a dimensionless graph was prepared that enables rapid evaluation of accumulated seepage versus time for trapezoidal channels with different water depths (fig. 27 in (5)).

The treatment of the transient seepage system as a two-dimensional infiltration problem is necessary only for the initial phases of the problem. As the seepage continues, gravity becomes the dominant factor, the flow system expands mainly in downward and very little in lateral direction, and the seepage rate begins to approach a constant value. For these later stages, the flow system can be analyzed as a succession of steady-state conditions. This procedure is described in (5) in which a dimensionless graph of the results for a trapezoidal channel is presented.

The transient systems discussed in the previous paragraphs refer to the condition of seepage into uniform soil with a relatively deep water table. Another type of transient seepage flow occurs when the ground-water table is relatively shallow and in direct contact with the water level in the channel. A change in the water level of the channel will then cause a change in the seepage (or drainage!). For example, it may be visualized that the channel is dry and that the water table is horizontal at the same elevation as the channel bottom. When water is admitted into the channel, seepage will cause the water table to rise, which in turn causes the seepage rate to decrease. This type of problem of transient flow can be analyzed with the Dupuit-Forchheimer assumption of horizontal flow ((5) and references therein). This assumption is valid only if the impermeable layer is relatively close to the channel bottom. The treatment can be extended to deeper positions of the impermeable layer by using an "equivalent" depth instead of the actual depth of the impermeable layer (5).

SUMMARY

Direct measurement of seepage can be obtained by inflow-outflow, ponding, seepage-meter, and salt-penetration techniques. The latter is a recently developed tracer technique whereby seepage is determined from the rate of advance of dissolved salt in the bottom material. The advantages and disadvantages of the various techniques are discussed. Another approach for obtaining quantitative seepage information is to calculate the seepage rate from a knowledge of the hydraulic conductivity profile of the soil and the position of the ground-water table. Solutions can be obtained by mathematical analysis or by analog or model studies. Analyses by resistance network analog and the resulting dimensionless graphs for determining the seepage rate are discussed. In certain cases, it will be of interest to know seepage in relation to time after water has entered a dry channel. This is essentially a problem of two-dimensional infiltration; it shows how simplified solutions can be obtained.

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FIELD EVALUATION OF SEEPAGE MEASUREMENT METHODS[1]

C. E. Brockway and R. V. Worstell[2]

INTRODUCTION

Irrigation project design, operation and maintenance, and canal-lining research and development require accurate and economical measurements of seepage rates. Drastically new methods for measuring seepage have not been developed, so existing field methods must be used. Each of these methods warrants an evaluation of its capabilities and limitations. This paper relates experiences with ponding tests, seepage meters, and inflow-outflow methods of measuring seepage from canals.

The results reported here represent the combined efforts of the University of Idaho Engineering Experiment Station, the Agricultural Research Service, and the U.S. Bureau of Reclamation.[3]

The study was performed in 1965 and 1966 on a 4.5-mile reach of the A and B Irrigation District Main Canal near Paul, Idaho. This canal is a part of the Minidoka Project of the U.S. Bureau of Reclamation. It is 25 to 30 feet wide with a gradient of about 0.5 feet per mile and flows at a depth of 5 to 5.5 feet during the irrigation season. Soils throughout the test reach are very uniform and consist almost entirely of Portneuf silt loam. A compacted, slightly cemented silt layer from 12 to 24 inches thick intersects the canal cross section throughout most of the the test reach. The flow system beneath the entire test reach is under tension gradients due to an impeding layer near the soil surface of the canal cross section.[4] Devices for recording water measurement were installed by the Bureau of Reclamation at the inlet and outlet and at all turnouts on the reach. A water budget for the irrigation season was maintained on this reach for 3 years, and the loss rates for 2-week periods were computed.

PONDING TESTS

Ponding tests were made on 1.5 miles of the test reach, 1 mile in the fall of 1965 and an additional ½ mile in the fall of 1966. The purpose of these tests was to measure actual canal seepage loss rates to use in determining water distribution efficiency on this part of the Minidoka Project, and to serve as standards for comparison with other seepage measurement techniques.

Plastic-covered earth dikes or plastic-covered wood bulkheads were used to isolate 0.5-mile-long ponds in each series of tests. Water stage recorders and hook gages were installed in corrugated metal stilling wells to measure water surface elevations. Recorders were located at each end of each pond to account for wind effects on the water surface elevations. In each of the ponding runs, the ponds were filled at least 12 hours before measuring the water surface drop began. Two runs were then performed, and the seepage rates on the second run were used as the operational seepage loss rates.

Ponding a 1-mile section of this canal with these techniques costs about $3,000.

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[1] Joint contribution from the University of Idaho Engineering Experiment Station and the Northwest Branch, Soil and Water Conservation Research Division, Agricultural Research Service, U.S. Department of Agriculture.

[2] Assistant research professor, Civil Engineering, University of Idaho, stationed at Kimberly, Idaho; and agricultural engineer, Snake River Conservation Research Center, Kimberly, Idaho.

[3] The contribution from the University of Idaho Engineering Experiment Station was supported in part from a U.S. Bureau of Reclamation cooperative agreement, in part from the University's "Short-Term Applied Research" program, and in part from funds provided by the U.S. Department of Interior, Office of Water Resources Research, as authorized under the Water Resources Research Act of 1964.

[4] Worstell, R. V. and Brockway, C. E. Estimating seasonal changes in irrigation canal seepage (1968). Paper presented at 1967 annual meeting of the Pacific Northwest Region of the Amer. Soc. Agri. Engin., Spokane, Wash. (Submitted in February 1968 for publication by A.S.A.E.)

The canal must be out of service for 10 to 14 days. This usually prohibits the performance of tests on main canals during the irrigation season. Tests early or late in the season in cold climates may require antifreeze in the stilling wells to prevent freezing. Regardless of these problems, ponding is still the standard method of measuring seepage losses. Whether it duplicates the operational seepage loss rates is open to question. The reasons why ponding seepage rates may vary from operational rates are discussed under the section "Seepage Meter and Ponding Test Results."

SEEPAGE-METER TESTS

Tests were run with a variable-head seepage meter developed by the Agricultural Research Service in the ponded reaches of the Main Canal before the ponding tests. In 1965, 71 tests were performed in one of the 0.5-mile ponds; and in 1966, 60 tests were performed in a 0.5-mile reach that was later ponded, and 26 tests in a 0.5-mile reach that could not be ponded because of a bulkhead failure. Tests were taken across the canal bottom at stations about 400 feet apart along the reach. Two groups of five measurements were made at each station.

Two men easily operated two ARS seepage meters in the 25- to 30-foot-wide canal. One man moved and inserted the meters in the canal, while the other recorded timed readings of manometers on the canal bank.[5] Two experienced men performed about 40 tests per day when seepage rates only were measured. The procedure for estimating hydraulic conductivity requires additional time, and about 20 tests per day were performed. The water level was maintained at about 22 inches at the centerline so that the meter could be inserted manually. A small flow was maintained to carry away sediments disturbed during the meter installation. Data were recorded on sheets with a punch-card format for processing with a digital computer.[6] Cost for obtaining a reasonable estimate of the seepage rate at a low water depth in this canal is about $300 per mile. When using this meter, water depths are limited to less than 2 feet, even though the operating depth of the canal may be much greater. A reasonable prediction of the seepage rate at the canal operating depth depends on the knowledge of the seepage flow system and soil conditions beneath the canal cross section.

SEEPAGE METER AND PONDING TESTS RESULTS

TABLE 1. — Comparison of seepage rates obtained by ponding and by seepage meter[1]

Item	1965	1966	Average 1965-1966
Average C_L water depth (inches)	22	20	21
Number of tests	71	60	66
Wetted area tested (percent)	.092	.086	.089
Ponded seepage rate (c.f.d.)	.50	.56	.53
Seepage meter rate (c.f.d.)	.68	.69	.68
Difference (c.f.d.)	.18	.13	.16
Difference (percent)	36	23	30

[1] Values for ponded rates are extrapolated down to the average level at which the seepage meter rates were measured.

Table 1 shows a comparison of seepage rates obtained from ponding tests and those estimated by seepage meter tests. Rates measured with the seepage meter are for an

[5] Bouwer, Herman, and Rice, R. C. Seepage meters in seepage and recharge studies. Jour. Irrig. and Drain. Div., Amer. Soc. Civ. Engin. Proc. 89 (IR 1): 17-43. 1963; and U.S. Department of Agriculture. Basic instructions for falling-head seepage meter technique. Agr. Res. Serv., Water Conserv. Lab. Rpt. 1, 11 pp. 1964.

[6] Brockway, C. E., and Worstell, R. V. Groundwater investigation and canal seepage studies. Idaho Engin. Exp. Sta. Prog. Rpt. 2. 1967.

average centerline water depth of 21 inches, while the canal operating depth was 5 to 5½ feet. The ability of the meter to accurately reflect actual seepage rates under similar conditions is evident. The meter rates in both instances are about 30 percent higher than the corresponding ponded rates. The ponded seepage rate could possibly be lower than that of an operating canal if suspended sediments and algae tend to settle to the bottom and partially seal it under conditions of zero velocity. A 30-percent error in estimating the seepage rate in a canal with low losses may not be economically important. However, a 30-percent error in the estimate for a canal with a higher seepage loss could result in an erroneous justification of a lining program.

Another reason for the 30-percent difference between the ponded rate and the seepage meter rate may be the difference in location of the water surfaces during the tests. The water-surface slope during the seepage meter tests was essentially equal to the friction gradient, or about 0.9 foot per half-mile. The ponded rate used for this comparison is computed at a level water surface elevation corresponding to the average elevation of the sloping water surface during seepage-meter tests. The actual seepage area for the ponded condition is not identical to the area sampled by the seepage meter tests. An estimate of the magnitude of the difference attributable to this effect is difficult. The meter itself may not measure the true seepage rate of the soil into which it is inserted.

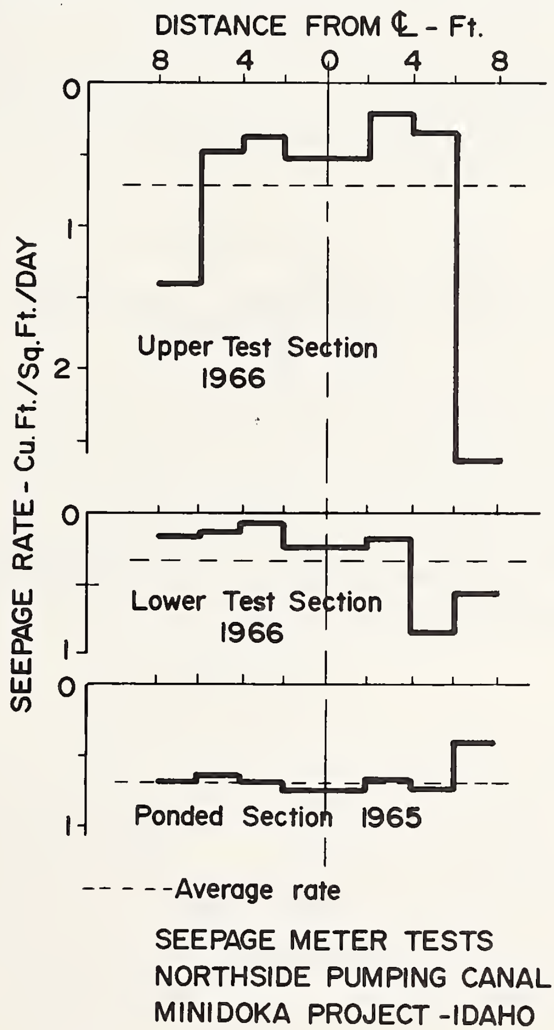

Figure 1.—Variation of seepage rate in cross section.

Disturbance of the soil during insertion of the meter bell can cause indicated seepage rates to be higher than actual. Inserting the bell may cause a "crack" through the restricting layer. An insufficient seal between the bell and the soil can also cause errors in measurement. However, with the ARS meter, the seal is always checked before each measurement (see ARS Water Conserv. Lab. Rpt. 1 listed in footnote 5). Warnick showed that with a constant-head-type meter, stepping on or pushing the meter bell in by hand caused measured seepage rates to be as much as 23 percent greater than the ponded rate.[7]

Differences in seepage rates across the channel were detected with this meter. In the 1966 tests, employees of the irrigation district shaped both sides of the upper pond before the seepage meter tests. This process removed the berm and disturbed the impeding layer on the side slopes, but did not disturb the canal bottom. In the lower pond, only one side of the cross section was shaped. Figure 1 shows the variation of seepage rate in the cross section for the three test reaches. The measured seepage rate should be lower near the outer edges of the wetted area where the effective water depth is less. The 1966 tests indicated higher seepage rates in areas where the impeding layer was disturbed. The comparison was tested statistically to assure that it existed.[8] This difference in rate within the cross section was not apparent for the 1965 tests which were made before shaping operations. Small differences in rates occurred throughout the length of each test reach, but no apparent physical reason could be found for this variation. Attempts to extrapolate 1966 measured seepage meter rates to operating depths were unsuccessful. Estimates using the measured hydraulic impedance of the restricting layer were not possible because the layer had been disturbed on the side slopes.

The seepage rate measured by the meter in the 1965 reach and extrapolated to operating depth was 1.42 c.f.d. This was compared to a seepage rate ponded at operating

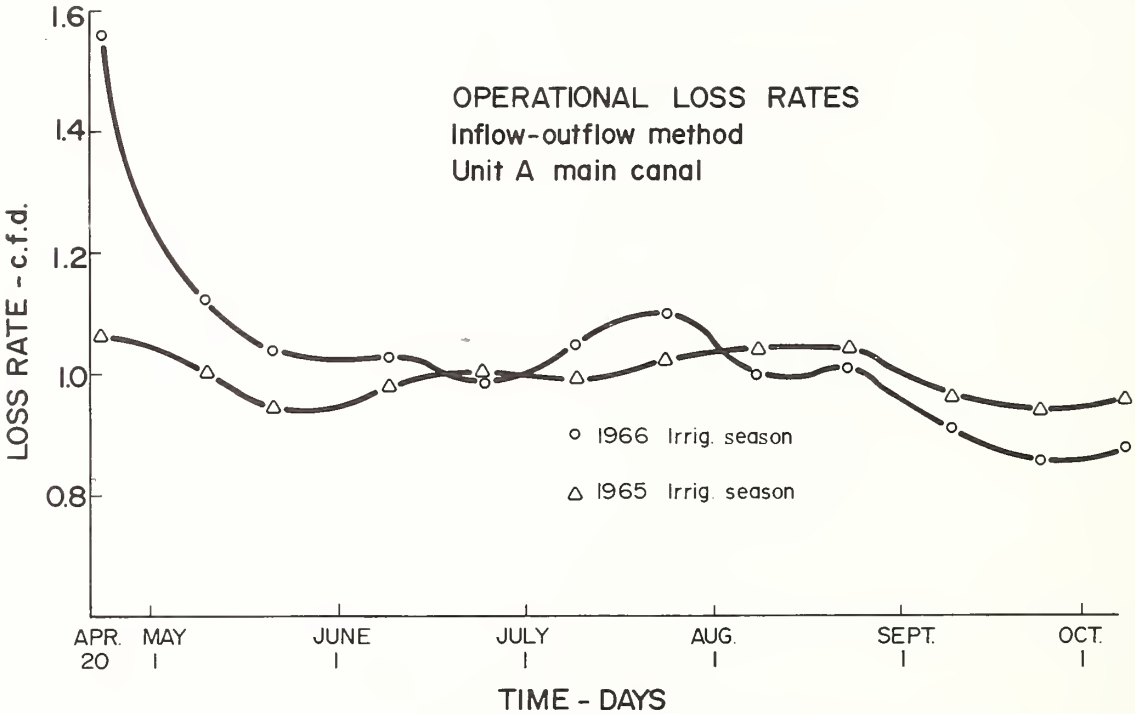

Figure 2.—Operational loss rates inflow-outflow method Unit A Main Canal.

[7] Warnick, C. C. Problems in seepage evaluation and control. U.S. Dept. Agr., ARS 41-90, Seepage Symposium Proc. 1963: 132-137. 1965.

[8] Ostle, Bernard. Statistics in Research. Ed. 2, Iowa State Univ. Press, Ames, Iowa. 1963.

depth of 0.67 c.f.d. The difference could be caused by the consolidated silt layer in the canal cross section, or, because of insertion of the seepage meter, the sealing layer could have been disturbed, particularly along the wall of the seepage meter. The measured hydraulic impedance in that case would be less than the actual value.

The inability to estimate operating level seepage loss rates by extrapolating measurements made with the ARS meter tends to counteract the advantages of its ease of operation and apparent accuracy. Further studies are underway to develop a meter using the variable head principle which can be used efficiently in an operating canal.

INFLOW-OUTFLOW MEASUREMENTS

The Bureau of Reclamation, as part of a study of water use on federally irrigated projects, instrumented the 4.5-mile reach of the Main Canal to determine operational losses. This system is one of the best installations of this type which has been made. Losses in the total reach length were computed for 2-week periods during the irrigation season and expressed as cubic feet per square foot per day (c.f.d.) over the entire wetted area. This loss rate is not a seepage rate per se, but includes other operational losses. A comparison of loss rates over the 1965 and 1966 seasons is shown in figure 2. Definite seasonal fluctuations are evident and are reasonably repeatable for the 2 years. The loss rates were considerably higher than the ponding seepage rates of 0.60 to 0.70 c.f.d.

Determining loss rates by inflow-outflow methods is usually very costly and probably should not be used solely to estimate seepage losses. The installation on the Unit A Canal involves over 20 recording flow-measuring devices, which are costly installations. With a large number of flow-measuring devices, the probable error in the estimate of loss rates can be quite large. Any error would be more significant in reaches with low loss rates.

ESTIMATING REQUIRED NUMBERS OF SEEPAGE METER TESTS

The following procedure can be used to estimate the number of seepage meter tests required to obtain a reasonable average value of the seepage rate from a reach of a canal.

A number of assumptions are required in the analysis: (1) The locations of measurements in the canal cross section were randomly selected, (2) the individual measurements were performed by competent personnel using the proper technique, (3) variability of the soils in which the known data sites were obtained approximated the variability of all other soils encountered, and (4) the distribution of seepage rates is normal. A level of confidence to be used in seepage meter tests can be defined as that which is based only on the variability of individual measurements as affected by random variation of soils and human techniques.

Using a Student's t distribution, the confidence interval for the mean is defined by

$$\overline{X} - \mu = \frac{ts}{\sqrt{N}} \qquad (1)$$

where

$\overline{X}$ = observed sample mean, or the mean of a number of seepage meter tests,

μ = population mean, or the mean of all possible tests,

s = sample standard deviation,

t = probability function which is dependent on the desired confidence level and the number of tests,

N = the number of tests.

Expressing the confidence interval as a percentage of the computed mean $\overline{X}$:

$$\frac{D\overline{X}}{100} = \frac{ts}{\sqrt{N}} \qquad (2)$$

where

D = maximum percent by which the computed mean might vary from the true mean at a given probability level.

For a selected value of D and estimated values of s and $\overline{X}$, the required number of tests, N_c, can be estimated by

$$N_c = \left(\frac{100\ ts}{D\overline{X}}\right)^2$$

or

$$N_c = \left(\frac{CVt}{D}\right)^2 \tag{3}$$

where $CV = \frac{100s}{\overline{X}}$ of the percent coefficient of variation.

Equation 3 must be solved by trial and error since t is a function of N_c.

With a selected 90-percent confidence level, a D of 20 percent, and estimated CV, equation 3 will give the number of tests required so that 9 times out of 10 the average of the group of seepage meter measurements will be within 20 percent of the true mean.

If individual seepage meter measurements are biased and do not accurately represent the seepage rate at a point, the sample mean and the true mean will be biased in the direction of error. The true mean is then not necessarily equal to the true seepage rate. Determining the accuracy of the seepage meter tests is possible only by comparing the computed mean with actual seepage rates determined by ponding.

Equation 3 can be solved graphically using figure 3, which was computed for a confidence level of 90 percent. Initial estimates of N_c can be made based on estimated values

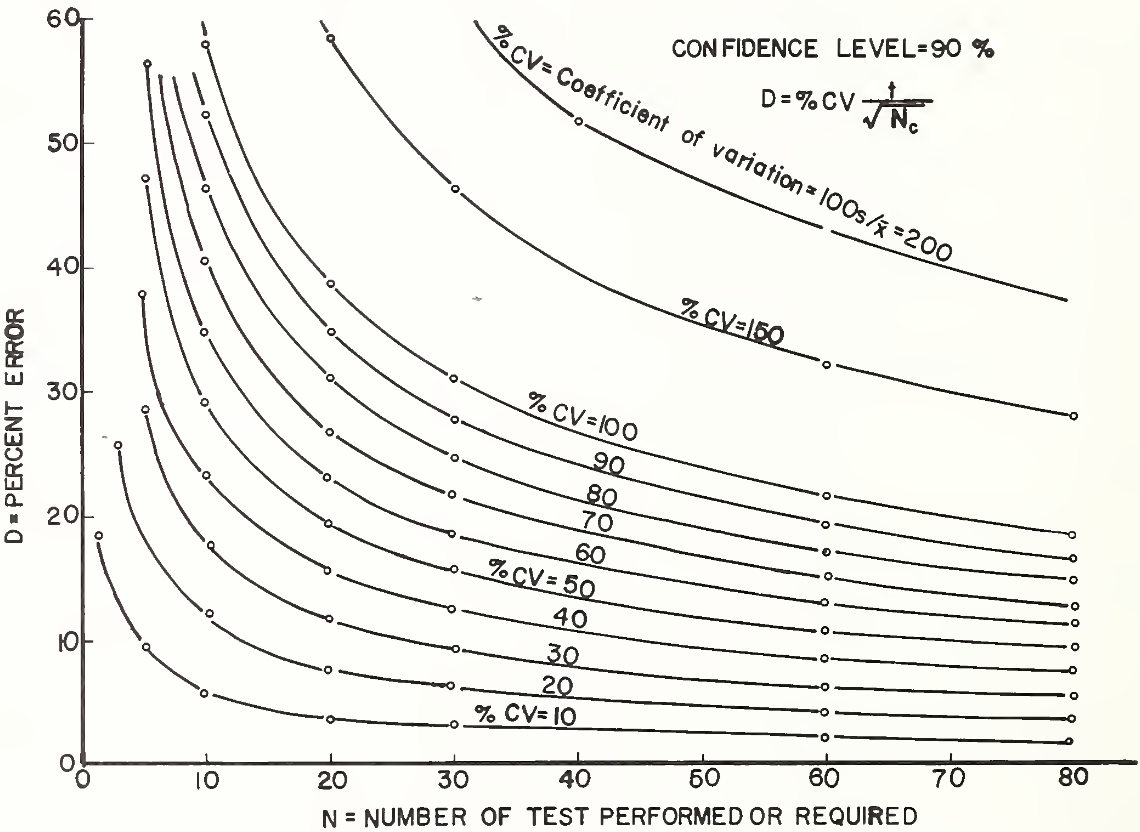

Figure 3.—Seepage meter study — determination of required number of tests.

of s and $\overline{X}$ and the computed value of CV. After a number of tests are run, figure 3 can be used to obtain a new estimate of N_c based on computed values of CV.

An average seepage rate can usually be estimated by examining the soil type and canal geometry. Past results of seepage meter studies can be used to obtain estimates of s. In the seepage meter tests with the ARS meter in 1965 and 1966, the average standard deviation of 17 groups of tests for a total of 156 tests was 0.538 c.f.d. A similar analysis of 54 sample groups for a total of 762 tests run with the U. S. Bureau of Reclamation meter on various types of soils showed an average standard deviation of 0.508 c.f.d.[9] A reasonable initial estimate of the standard deviation is probably about 0.5 c.f.d.

TABLE 2.—Statistical analysis of seepage meter tests, Northside Pumping Canal Test Section, 1965

Item	Initial estimate	Number of tests completed 10	20	30	40	51	61	71
Mean seepage $\overline{X}$ (c.f.d.)	0.75	0.727	0.658	0.614	0.661	0.643	0.693	0.673
Standard deviation s (c.f.d.)	.5	.481	.410	.357	.364	.387	.448	.432
Coefficient of variation (%)	66.7	66.2	62.4	58.2	55.1	60.1	64.6	64.2
Tests required N (No.)	32	31	28	25	22	26	30	29
Actual D (%)	—	39	24	18	15	14	14	13

Table 2 is an example of the use of this procedure for obtaining an initial estimate of N_c and then revising the estimate after a number of tests have been obtained. Estimates of the seepage rate for the Portneuf silt loam soil in the 0.5-mile reach of the canal varied from 0.5 to 1.0 c.f.d., so an initial value of 0.75 was chosen for $\overline{X}$. The initial values in table 2 are based on a confidence level of 90 percent and a D value of 20 percent. For this series of tests, the required D was obtained after about 30 tests and the testing could have been terminated. Similar analysis for confidence limits other than 90 percent could be made using equation 3, or curves similar to figure 3.

DISCUSSION AND CONCLUSIONS

Of the available methods for evaluating seepage losses, the ponding test is the most accurate but the most expensive. The use of seepage meters for obtaining estimates is fast and economical. However, new types of meters capable of functioning in canals at operating depth should be studied. Almost all the available meters are capable of measuring seepage with reasonable accuracy at a point, but discretion must be used in the amount of confidence placed in average values determined from meter tests. The procedure outlined for estimating the number of meter tests required can be used to judge the confidence to be placed in any group of tests.

Inflow-outflow methods are usually too expensive to be used for short-duration seepage measurements. However, a good installation does indicate seasonal changes in loss rates. Accuracy of inflow-outflow determinations is limited by the flow-measuring devices, but for canals with large seepage losses, inflow-outflow methods may be the most expedient and sufficiently accurate.

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[9] Engr, P. 'F.' January 1965. Memorandum to E. J. Carlson, Special Investigations Section, Hydraulic Laboratory, U.S. Bureau of Reclamation, Denver, Colo.

SEEPAGE DETECTION BY REMOTE SENSING[1]

Thomas R. Ory[2]

INTRODUCTION

A decade ago, aerial-reconnaissance equipment consisted of a variety of aerial cameras, a few field sensors such as magnetometers, and some experimental radars and infrared scanners developed by military agencies. By the time the last Seepage Symposium was held, 5 years ago, the term "remote sensing" was being heard with increasing frequency and a vast array of new sensors had been conceived, fabricated, and tested. Now, 5 years later, it is possible to remotely measure the properties of earth materials in spectral regions extending from gamma rays to audio frequencies. The term remote sensing has generally become associated with any type of data recorded by a sensor which measures energy emitted or reflected by objects located at some distance from the sensor. Usually, but not always, the sensors are carried by an aircraft or, more recently, a spacecraft. This discussion will be limited to those remote sensors, other than cameras, capable of being utilized in aircraft.

TABLE 1. — Electromagnetic spectrum

Electromagnetic spectrum	Wavelength	Sensors
Radio	300 m. —	Audio, VHF
Microwave	.3 cm. — 3 m.	Radars Microwave radiometer Scatterometer
Infrared	1 — 30μ	Infrared scanner Infrared radiometer
Visible and near IR	.4 — 1μ	Multispectral cameras Vidicons
Ultraviolet	300Å — .4μ	UV scanners
X-ray	3Å	— — —
Gamma ray	.03Å	Scintillometer
Force fields:		
Magnetic		Magnetometer
Gravity		Gravimeter Gradiometer

Table 1 illustrates the electromagnetic spectrum with labels on the primary regions of interest in remote sensing, the wavelength limits corresponding with these regions, and the types of sensors available for remote measurements of the energy in each region. In addition, the primary force fields and corresponding sensors are shown.

[1] Contribution from H. R. B. Singer, Inc. A subsidiary of the Singer Company.

[2] Director, Radiometrics Laboratory.

TABLE 2. — **Remote sensor characteristics**

Sensors	Mode	Fundamental property measured
Audio, VHF	Active	Conductivity
Radars	Active and passive	Roughness, dielectric constant
Microwave radiometer		
Scatterometer		
Infrared scanner	Passive	Temperature and emissivity
Infrared radiometer		
Multispectral cameras	Require light source	Spectral reflectance
Vidicons		
Ultraviolet scanners	Require light source	Reflectance and flourescence
Scintillometers	Passive	Radioactivity
Magnetometer	Passive	Magnetic field strength
Gravimeter	Passive	Density
Gradiometer		

Table 2 tabulates the primary remote sensors presently available along with their operating modes and the primary physical property which they are capable of measuring. In general, the capability for good weather penetration and subsurface data acquisition is confined to the low-frequency end of the spectrum — the radio and radar end. On the other hand, the capability for high-spatial resolution is much better in the infrared and visible regions, and unique penetrating capabilities are available in the very high frequency X-rays and gamma rays.

REMOTE SENSOR PROBLEM APPROACH

To determine which remote sensor or combination of sensors has the best probability of success on a particular data collection problem, we must first identify the basic properties of the problem. In seepage detection, it is necessary to define seepage in terms of physical parameters which can be related to remote sensor capabilities.

First, seepage normally occurs with a definite spatial distribution; that is, seepage from a canal or river normally distributes laterally a finite distance from the channel and therefore, the search may be confined to a relatively narrow corridor in the vicinity of the channel.

Second, a porous and permeable subsurface medium is required in order for seepage to occur.

Third, the following physical properties of the seepage conductor are changed by the seepage of water through it:

temperature

moisture content

density

conductivity

dielectric constant.

A number of important parameters for the design of a remote sensor approach to the problem of seepage detection have now been identified.

The area of search has been identified as a relatively narrow corridor whose dimensions can be easily obtained from maps.

In addition, we are looking for zones within this narrow corridor in which the conditions necessary for seepage occur; namely, an intersecting porous and permeable medium.

Finally, we have identified a number of physical properties which can be remotely measured to determine that seepage is, in fact, occurring.

SENSOR SELECTION

Correlating the physical changes caused by seepage with the measuring capabilities of available remote sensing techniques yields several potentially applicable sensors as

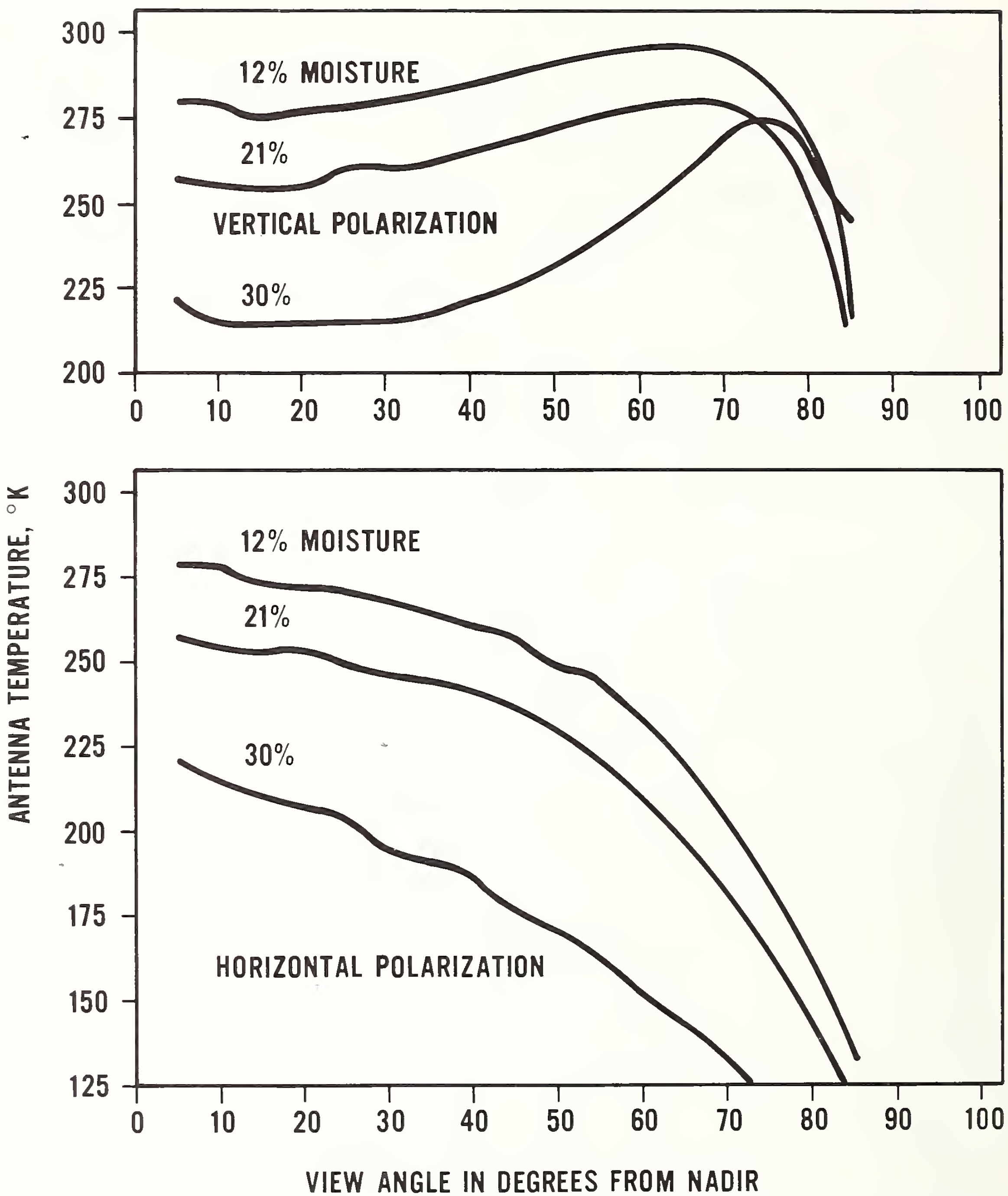

Figure 1.—13.5 GHz radiometric temperatures of playa sediment with variable moisture. (Courtesy, Remote Sensing Group, Ryan Aeronautical Co.)

follows:

Physical Change	Sensor
temperature	infrared scanner or radiometer
moisture content	radar, microwave radiometer, scintillometer
density	gravimeter
conductivity	HF or VHF technique
dielectric constant	radar or microwave radiometer

Of the sensors which appear to have application in the detection of seepage, infrared scanners and radars are the only sensors presently capable of producing two-dimensional maplike records with reasonably good spatial resolution. Because of the need to relate seepage to a source and to accurately determine its distribution, the inability of the other sensors to produce a maplike record is a serious limitation. A single line of remote sensor data representing terrain properties along the flight track of the aircraft is probably of very little value in the detection and evaluation of seepage problems. Moreover, attempting to provide two-dimensional data by placing flight lines at spacings closer than about ¼ mile is impractical, and this spacing normally would be too gross to provide a good seepage evaluation. Therefore, infrared scanners and radars appear to be the most likely remote sensors to supplement aerial photography in the detection and evaluation of seepage problems at the present time.

APPLICABLE REMOTE SENSOR RESULTS

Both radar and infrared devices have been investigated from the standpoint of their application to the remote detection of soil moisture. The applicability of a radar device to the problem of soil-moisture measurement depends largely upon the frequency of the radar energy. Figure 1 illustrates the response of a microwave radiometer to playa sediments of differing moisture content. A microwave radiometer is a very sensitive passive radar receiver, capable of detecting and precisely measuring the intensity of specific frequencies of electromagnetic energy. The illustration demonstrates the ability to distinguish relatively dry from relatively wet soils at a frequency of 13.5 GHz. Lower frequencies would likely be more affected by gross changes such as the top of the water table, while higher frequencies would record surficial characteristics such as roughness and vegetation.

Because of the large soil temperature differentials caused by anomalously high-water content in arid and semiarid areas, infrared devices have been particularly attractive for use in these areas. Figure 2 is an infrared image of a segment of the Coachella Canal

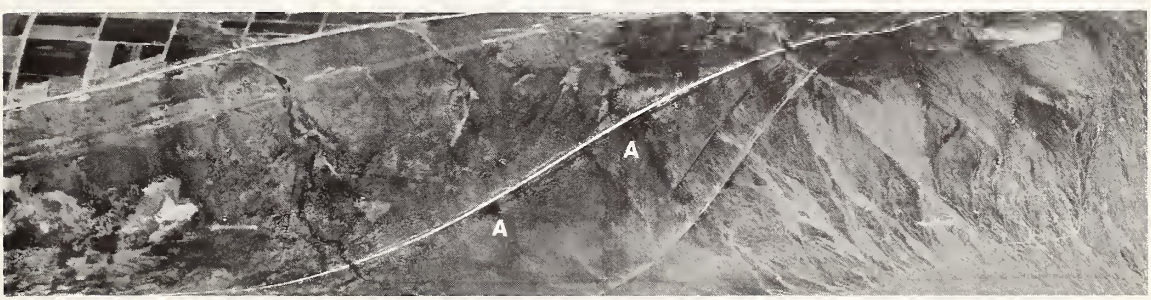

Figure 2.—Infrared image of a section of the Coachella Canal, Imperial Valley, Calif.

in the Imperial Valley of California. This image was generated at about 3:00 a.m. using an infrared detector operating in the 8-14μ spectral region. This relatively high altitude view of the canal and its surroundings illustrates the potential of infrared techniques for the detection of moisture anomalies. The two zones marked "A" are cool areas caused either by seepage from the canal or ponding of groundwater along the canal embankments.

Figures 3 and 4 show similar nighttime infrared images of canals in Southern Alberta, Canada. Note that very cool apparent temperatures are recorded for some

Figure 3.—Infrared imagery showing canal seepage in Southern Alberta, Canada. (Courtesy Alberta Department of Agriculture.)

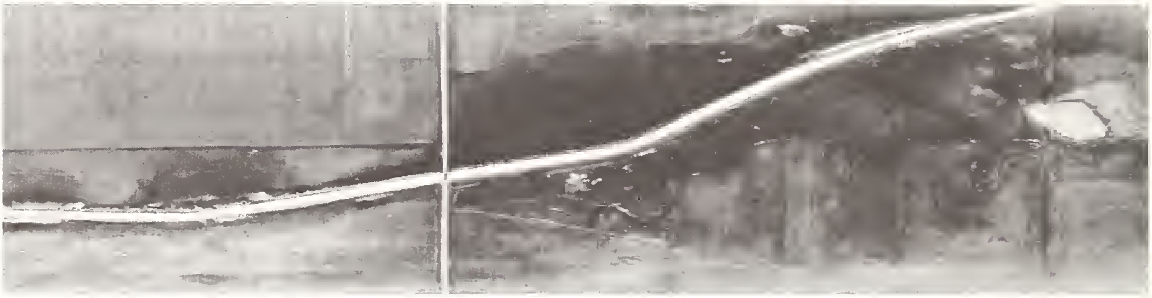

Figure 4.—Infrared imagery showing canal seepage in Southern Alberta, Canada. (Courtesy Alberta Department of Agriculture.)

distance on either side of the canal segments which are imaged. These signals emanate from areas in which canal seepage has raised the groundwater level to the point at which salts are deposited on the surface through evaporation. These salt-covered areas reflect solar radiation during the day and therefore appear much cooler than the surrounding terrain at night. The cooler temperatures are also caused by evaporation of water that has seeped from the canal to adjoining fields. In areas where the salt crust is not present, the relative coolness caused by evaporation provides a rough measure of the degree of wetness of the soils. Similar cool temperatures have been recorded by infrared scanners in areas where groundwater is confined to aquifers many feet below the surface, and the cool surface temperatures are apparently due to the evaporation of water which rises to the surface through capillary action.

Neither radar nor infrared measurements should be construed to be absolute indicators of seepage. Both techniques produce ambiguous data whereby natural processes other than seepage could give rise to similar, if not identical, measurements. However, it has been demonstrated that both techniques have potential in providing for the detection of seepage through aerial observation. It is recommended that conventional or color aerial photography be used as a reference standard for radar and infrared to reduce ambiguity.

THE FUTURE

Extensive evaluations of remote sensors are presently being undertaken by a number of agencies. Several other remote sensors have demonstrated capabilities which may prove to be valuable for seepage detection. For example, although gamma-ray detectors measure radioactivity, the emission of gamma rays is significantly suppressed in areas which are waterlogged so that this technique might be used as an indirect indicator of seepage. Some radio frequency techniques used to search for conductive ore bodies also demonstrate the ability to detect areas of slightly higher conductivities caused by high-moisture contents. In addition, modification and more precise calibration of radar and infrared sensors may provide the capability for estimating the rate of seepage in the near future. Thus, the potential for valuable contributions to the problem of seepage detection and evaluation through remote sensing is high. Moreover,

the potential savings which may be realized by faster and more reliable techniques for seepage detection should stimulate additional research and result in the availability of new techniques in the near future.

SUMMARY

A number of remote sensing techniques have been shown to have application in the detection and evaluation of seepage. Radar and infrared sensors are presently available for use in seepage detection research and, to a limited extent, for operational work. When these sensors are used after a careful analysis of the detection problem, and with utilization of the information available from conventional techniques, the success probability of remote seepage detection should be very good. Future research will undoubtedly yield less expensive and more reliable remote sensors for this application as the payoffs from initial employment begin to stimulate their use.

ECONOMIC COMPARISONS OF OPEN CONDUIT AND PIPE IRRIGATION DISTRIBUTION SYSTEMS[1]

B. A. Prichard[2]

Recently, a water users' association presented the following resolution to the Bureau of Reclamation concerning a potential irrigation development:

"Be it resolved that the Bureau of Reclamation and other agencies be encouraged to consider the following points in the original plans for construction and development of the irrigation project . . . :

"1. To construct the main canal and laterals in such a manner which will prevent seepage and waste water to reduce drainage problems.

"2. That the system deliver water to the farms under adequate pressure to encourage sprinkler irrigation for better water control to help prevent erosion and drainage problems.

"3. That as much of the system as possible be underground with a minimum amount of land taken out of production and for better weed control and also to permit farms to be laid out in squares or rectangles."

Of course, these are aspects of project planning that are given thorough consideration by the Bureau irrespective of such a resolution. But the resolution indicates the awareness even of potential irrigators of the seepage problems that are being discussed at this symposium. It also highlights some other considerations which are related to the solution of seepage problems. The association recognized that canal linings help reduce seepage problems, and it recognized that seepage problems are also related to poor irrigation practices. Further, the suggestion that as much of the system as possible be placed under ground recognizes the economic limitations of its proposal while pointing out some of the advantages that accrue only through a pipe system. The economic limitations for canal linings is our topic for consideration. In this discussion, the term "canal linings" is intended to include pipe. The financial limitations are equally important, but are not treated here. At the first seepage symposium, Charles Welker[3] stated that in West Pakistan "Lining of operating canals was not economically feasible in the early days of irrigation development. Water was sufficiently abundant and the cost of the canal systems so low that the additional revenues which would have been realized from the lost seepage could not . . . justify the cost of lining to reduce the seepage loss." The same statement could be made about some of the early irrigation developments in the Western United States. But generally, this is no longer true, because such instances are becoming quite rare. In planning new projects, the Bureau of Reclamation now requires full justification for plans that do *not* provide for lining the canals. To conserve water and to secure other benefits, planners must consider fully the lining or placing in pipe of all constructed waterways for the conveyance and distribution of project water supplies.

To facilitate the economic evaluation of lining of canals and laterals, a study was made by the Bureau of a sample area and the results have been published in a report.[4] The report presents guidelines and illustrates a procedure. It does not answer such specific questions as what size or capacity of lateral may be economically placed in pipe or which type of lining is the most economical. But it does present some general conclusions which are helpful in making more specific determinations. These conclusions are:

1. The value of water conserved by the addition of canal lining is generally the most significant factor in determining economic justification for a canal lining.

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[1] Contribution from Division of Irrigation Operations, Bureau of Reclamation, U.S. Department of Interior.

[2] Engineer, Bureau of Reclamation, Denver, Colo.

[3] Welker, C. H. Seepage problems in West Pakistan. U.S. Dept. Agr. ARS 41-90, Seepage Symposium Proc. 1963: 31-38. 1965.

[4] U.S. Department of the Interior. Economic justification for canal lining in irrigation distribution systems. 1965.

2. In areas where there is an excess of irrigable land for the available water supply or where there are other potential uses for the water, canal linings generally will be economically justified.

3. Other substantial savings in cost or increases in tangible benefits which may result from canal lining include:

a. Reduction in annual operations, maintenance, and repair (OM&R) costs
b. Reduction in investment costs for the storage system
c. Reductions in investment costs for the drainage system
d. Reduced right-of-way costs
e. Increased benefits from farmed rights-of-way for pipe systems

4. Certain other benefits that may have real value but that are generally not susceptible of evaluation in formal analyses include:

a. Improved safety
b. Reduction in spreading noxious weed seeds
c. Reduction in weed-chemical contamination of water
d. Improved farm appearances

To illustrate the relative importance of these conclusions, it will be helpful to examine some of the results from the sample area study. Table 1 compares the economic analysis of a system with unlined canals and laterals with analyses of two alternative lined systems. The first alternative lined system has pipe laterals; the second, concrete-lined laterals.

TABLE 1. — Economic analyses of unlined—versus lined—distribution systems

Item	Unlined canal, unlined laterals	Concrete-lined canal, pipe laterals		Concrete-lined canal, concrete-lined laterals	
	Total	Total	Increment	Total	Increment
Acreage served	10,693	15,344	4,651	15,344	4,651
Investment costs					
Construction costs, distribution system:					
Canals	$ 760,000	$ 1,566,000	$ 806,000	$ 1,566,000	$ 806,000
Laterals	1,122,000	4,701,000	3,579,000	2,324,000	1,202,000
Drains	749,000	1,074,000	325,000	1,074,000	325,000
Subtotal	2,631,000	7,341,000	4,710,000	4,964,000	2,333,000
Interest during construction	82,000	229,000	147,000	155,000	73,000
Investment costs, distribution system	2,713,000	7,570,000	4,857,000	5,119,000	2,406,000
Supply works, share of investment[1]	5,347,000	5,347,000	0	5,347,000	0
Total investment costs	8,060,000	12,917,000	4,857,000	10,466,000	2,406,000
Annual costs					
Annual equivalent of investment[2]	264,000	423,200	159,200	342,900	78,900
Annual OM&R, distribution system	27,500	28,500	1,000	34,500	7,000
Annual OM&R, share of supply works[1]	10,700	10,700	0	10,700	0
Total annual costs	302,200	462,400	160,200	388,100	85,900
Annual benefits					
Irrigation	834,100	1,196,800	362,700	1,196,800	362,700
Negative effects from right-of-way	4,600	3,100	(1,500)	6,600	2,000
Total annual benefits	829,500	1,193,700	364,200	1,190,200	360,700
Benefit-cost ratio	2.74 to 1.00	2.58 to 1.00	2.27 to 1.00	3.07 to 1.00	4.20 to 1.00
Excess of benefits over costs	527,300	731,300	204,000	802,100	274,800
Investment cost per acre	754	842	1,044	682	517

[1] Increases in acreage served by the alternative systems would normally increase the allocation of joint costs to irrigation; however, this increase in cost would not be significant and has been omitted in this analysis.

[2] Annual equivalent at 100 years and 3⅛-percent interest, factor 0.03276.

The unlined distribution system for the study area consists of 10.6 miles of main canal and 42.7 miles of unlined laterals serving 10,693 irrigable acres. Each of the comparative lined systems would serve 15,344 irrigable acres. The expanded acreage would be irrigated with water conserved through lining. For each alternative, the lined canal would be 15.2 miles long. The length of the open-lined laterals would total 61.2 miles, while only 54.2 miles of pipe laterals would be required.

The variables and assumptions that are elements of the study will be discussed, but first compare the benefits and benefit-cost ratios shown in the table. In this study, the lined canal with an open-lined lateral system has the most favorable benefit-cost ratio, 3.07 to 1. The pipe-lateral system has the lowest ratio. However, the excess of benefits over costs for the pipe-lateral system is greater than that for the unlined system by more than $200,000. In other words, benefits derived by use of the water conserved to irrigate additional lands would be sufficient to justify a decision to line the canals and place the laterals in pipe. The decision between an open-lined and a pipe-lateral system in this study would take into consideration the unevaluated benefits discussed previously — improved safety, reduction in spreading noxious weed seeds, reduction in weed-chemical contamination, and improved appearance.

An examination of the incremental annual benefits and costs reveals the importance of the value of water conserved. In this instance, with contiguous irrigable lands available, each acre of additional land irrigated would produce $78 of annual benefits. In this particular area, the diversion requirement at the canal heading for the unlined system was estimated to be 1.88 acre-feet per acre and for the lined system, 1.33 acre-feet per acre. By irrigating additional lands, the benefits per acre-foot of water were thereby increased from $41 to $58 per acre-foot. Benefiits per acre-foot of water for similar studies in other areas where additional irrigable lands are available will vary appreciably. A decrease in the amount of water conserved through lining or an increase in the diversion requirement per irrigable acre would cause a proportionate decrease in the additional acres which could be irrigated with the water saved. An increase or decrease in the annual benefits per acre of irrigated lands would likewise cause a proportionate change in benefits per acre-foot of water. For example, gross crop values on Bureau projects on a project-wide basis reported for 1966 varied from about $20 to nearly $1,000 per irrigated acre.

If additional irrigable lands are not available, there may be other uses or markets for the conserved water. The analysis then should recognize benefits derived from the alternative uses. Where other project uses for the conserved water do not exist, the size and costs of the storage and distribution system would be reduced to recognize the reduction in the amount of water lost through seepage.

The most significant tangible benefit next to value of water conserved generally is reduced operation, maintenance, and replacement costs. In this study, it was estimated that the OM&R costs for the pipe system would be about $0.70 per irrigable acre less than costs for unlined system; and that the costs for the open-lined system would be $0.32 per irrigable acre less than those for the unlined system. It is not likely that savings in OM&R costs will be greater than $1 to $2 an irrigable acre even though OM&R costs reported for Bureau projects vary from about $2 to about $30 per irrigable acre. The present worth of $1 per acre for 100 years is about $30. Considering that in this study the costs per acre of the pipe laterals are about $200 per acre more than the costs for unlined laterals, it becomes apparent that tangible savings in OM&R costs alone will not pay for the additional costs of pipe. Variation in OM&R costs and gross crop values from a few Reclamation projects are shown in table 2.[5]

The costs per acre of the drainage systems shown in table 1 are the same for each alternative. It was estimated in this instance that the drainage system designed for the project would adequately remove excessive seepage from the canals or laterals. Under a less favorable condition, an unlined system would require additional drains to remove canal seepage before it causes crop damage. An analysis of alternative lined or unlined systems, where drainage system construction costs can be reduced by providing lining, should also recognize the corresponding reduction in drainage system OM&R costs.

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[5] U.S. Department of the Interior. Summary report of the Commissioner, Bureau of Reclamation. Statistical Appendix Parts I, II, and III. Washington, D.C. 1966.

TABLE 2. — Typical OM&R costs and gross crop values per irrigated acre, 1965

State, project, division, irrigation district, or unit	Irrigated area	Gross crop value per irrigated acre	Total OM&R cost per irrigated acre
ARIZONA	Acres	Dollars	Dollars
Gila:			
North Gila Valley unit	5,956	450.63	6.63
Yuma Mesa unit	16,976	360.29	20.80
Salt River:			
Salt River Valley Water Users' Association	145,329	353.44	36.48
CALIFORNIA			
All-American Canal System:			
Coachella Division	59,890	806.43	22.90
Imperial Division	432,612	293.76	12.60
Orland	17,200	108.74	8.25
COLORADO			
Grand Valley:			
Garfield Gravity Division	20,755	113.27	5.71
Orchard Mesa Division	6,325	269.73	9.04
Smith Fork	8,456	46.46	1.61
Uncompahgre	63,883	94.02	4.51
IDAHO			
Boise:			
Arrowrock Division	140,162	132.58	6.03
Payette Division	52,795	112.81	6.74
KANSAS-NEBRASKA			
Missouri River Basin:			
Nebraska-Bostwick Irrigation District	19,826	122.97	4.68
MONTANA			
Sun River:			
Fort Shaw Division	8,003	24.57	3.02
Greenfields Division	61,780	41.60	2.70
MONTANA-NORTH DAKOTA			
Lower Yellowstone:			
Districts 1 and 2	45,840	100.81	3.72
NEBRASKA-WYOMING			
North Platte:			
Gering-Fort Laramie Irrigation District	51,502	107.21	2.72
Goshen Irrigation District	51,520	84.52	3.73
NEVADA-CALIFORNIA			
Truckee Storage	19,226	63.97	2.17
NEW MEXICO-TEXAS			
Rio Grande:			
Elephant Butte Irrigation District	83,259	266.67	6.71
El Paso County Water Improvement District No. 1	54,868	233.02	9.03
OKLAHOMA			
W. C. Austin	41,242	102.47	3.78

TABLE 2. — Continued

State, project, division, irrigation district, or unit	Irrigated area	Gross crop value per irrigated acre	Total OM&R cost per irrigated acre
OREGON	Acres	Dollars	Dollars
Owyhee:			
North Division	60,890	194.27	5.30
South Division	37,477	175.42	5.80
Vale:			
Vale Irrigation District	34,096	81.77	3.44
OREGON-CALIFORNA			
Klamath:			
Langell Valley Irrigation District	15,800	75.52	2.65
Klamath Irrigation District	33,221	114.90	5.40
SOUTH DAKOTA			
Belle Fourche	53,380	45.58	2.56
WASHINGTON			
Columbia Basin	410,645	173.06	8.11
WYOMING			
Missouri River Basin:			
Bighorn Basin Division: Hanover-Bluff Unit	6,810	120.07	5.30

In the analyses, adjustments for right-of-way requirements for the pipe system were considered in the construction costs estimate and in the annual benefits. Farming would be permitted over the pipe where operating roads are not required. However, the irrigable acreage shown in table 1 for the pipe-lateral system is the same as that for the lined-open-lateral system. The total irrigable acreage depends upon the available water supply. Therefore, the increased benefits which result from farming of the pipe right-of-way are only equivalent to the benefits for dryland farming of that acreage. In table 1, this benefit is treated as a reduction of the negative effects from right-of-way. Negative effects result from removal of dry lands from production and were estimated for this study to be $9.15 per acre.

The unevaluated benefits from placing laterals in pipe are, in many instances, significant to a decision. Some of the irrigation districts operating older Bureau projects converted many miles of their open systems to pipe. Sometimes the conversion was financed in part by individual farmers who recognize the unevaluated benefits. The Bureau also recognizes these benefits so that where the results of the economic analyses are reasonably close, a decision is made for pipe. The increasing emphasis on safety and on water quality causes these unevaluated benefits obtainable through pipe systems to be of increasing significance.

At the last symposium, R. J. Willson[6] discussed the Bureau's activities under the Lower Cost Canal Lining Program. Subsequently, the activities of that program have been revised to include closed conduit systems. Besides the search for economical materials and construction methods for closed conduits, there is a search for more data on the variables encountered in an economic analyses such as the one illustrated. Hopefully, further parameters and guidelines will be developed to simplify the procedures herein discussed.

[6] Willson, R. J. A lower cost canal lining program. U.S. Dept. Agr. ARS 41-90, Seepage Symposium Proc. 1963: 85-93. 1965.

THE INVESTMENT DECISION FOR SEEPAGE REDUCTION[1]

Robert A. Young[2]

INTRODUCTION

Those responsible for this program have asked me to discuss with you the question of "How do you determine when you will save money by investing in seepage reduction?" Expenditures undertaken for seepage control are standard examples of the investment problem, where expenditures are made at some point in time in anticipation of attaining a larger future stream of income. Thus I interpret that my assignment calls for a textbook discussion of the principles of investment decisions as these apply to seepage-reduction situations. Time permits attention to only a few of the major issues; however, a variety of sources on the theory of investment decision is available to the reader wishing to pursue the finer points.

A general framework for making optimal planning decisions calls for the following elements: *Criteria* for making choices; a set of *alternative courses of action or policies* proposed as solutions to the problem; and expected *outcomes* or *consequences* of each of the alternative policies. A formal model is usually developed which provides a systematic means of expressing the consequences of alternative policies in terms of the chosen evaluative criteria. In the following discussion, I will first discuss the criterion problem, then argue for the validity of a particular set of decision models for the seepage-investment problem, and finally discuss some pertinent issues arising when these models are adapted to the solution of such problems.

MODELS FOR INVESTMENT PLANNING

Criteria for Choice in the Investment Problem

The program committee, by their inclusion of my topic, have in effect postulated that economic criteria should be considered in making investment decisions concerning the issue in question. That the maximization of net income is the appropriate criteria for the private firm is not at issue. However, since many investments in seepage reduction are made by public and quasi-public agencies, often on the basis of unclear criteria, some additional discussion is in order.

Allen Kneese observed that the contribution of economics to the problem of public investment planning in the natural resources field is not widely understood outside the profession itself (*4*). Since his analysis is quite pertinent here, I will summarize for you his main points.

Values are preeminent in the planning process, and there must be some means of thinking about them systematically and if possible, quantifying them. The importance of economics to the process derives from the fact that the price and allocation theory of economics and its prescriptive application to problems of human welfare *is* a theory of social values. (Somewhat confusingly, this prescriptive theory is called "welfare economics.") It is a theory of social values sufficiently detailed, precise, and logical to assist in the derivation of decision criteria and to provide a structure for systematic quantitative measurement. Most important, the theory reflects the judgment that individual tastes and preferences are to govern the use of resources in a free society, a value judgment widely accepted in our culture.

Kneese quotes Gaffney's interpretation of welfare economics (*1*, p. 20).

"One of the most important functions of economic analysis is to evaluate public policy. Economics, contrary to common usage, begins with the postulate that man is the measure of all things. Direct damage to human health and happiness is more directly "economic," therefore, than damage to property which is simply an intermediate means

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[1] Contribution from the Agricultural Experiment Station, The University of Arizona, Tucson, Ariz.

[2] Associate professor, Agricultural Economics.

to health and happiness. Neither do economists regard "economic" as a synonym for "pecuniary." Rather money value is but one of many means to ends as well as a useful measure of value."

Kneese continues:

"Economic analysis serves an essential function in social decision-making, arbitrating among the rival claims of different interest groups and disciplines. There is little room in it for absolutism. This is why the economist tends to speak in terms of demands and preferences rather than requirements and needs. The basis upon which welfare economics establishes the commensurability of different goods and services is the willingness of consumers to pay for the resources necessary to produce them in view of the willingness of the same or other consumers to pay for the use of those resources in alternative employments. In this game, there are no external standards for a "good result." Rather, the desirable result is deemed to be the one that goes furthest in terms of satisfying human wants, given limitations on resources and the prevailing distribution of income."

If willingness of consumers to pay for goods and services is accepted as a basis for registering individual preferences, then the *net national income* may be used as a criterion for public expenditure evaluation. (Net national income may be interpreted as benefits minus costs, to whomsoever each of these might accrue.)[3]

To clarify the point further, we might examine an alternative criterion. It is often argued that in arid environments *water losses should be minimized.* This criterion is objectionable on the grounds that a physical entity (in this case, water losses) is being optimized. The entities or variables in the criterion function should be, insofar as possible, closely related to human needs and satisfactions. While less than an ideal criterion, maximizing net national income is preferable to any purely physical entity as a criterion for choice in public expenditures. It is easy to recognize that the criterion of minimizing water losses is never strictly observed in practice. Solution to the water transport problem by this criterion would always entail carrying water in the most impervious conduit available, a practice for which the incremental gains would seldom exceed the incremental costs.

Welfare economics is not claimed to be a perfected instrument for evaluating public policy. However, in the absence of a preferable alternative theory, I conclude that when costs and benefits are suitably represented, the conventional economic investment models, which are designed to maximize net income, are as proper and appropriate for planning decisions by public agencies as they are for private. I turn to a characterization of those models and their application to seepage-reduction evaluation.

The Economic Theory of Investment Decisions

Classical deterministic models are emphasized here, since the expenses of the more refined approaches apparently are not warranted for the relatively limited investments considered.

There are a number of formulas advanced for evaluating the economic desirability of proposed projects.[4] The preferred formulation is the "Present Value Rule" which calls for adoption of any project for which the present value of the associated stream of net benefits or net receipts, discounted at the appropriate interest rate, is greater than zero. Present value may be defined by the following formula.

$$V_o = s_o + \frac{s_1}{1+i} + \frac{s_2}{(1+i)^2} + \ldots + \frac{s_n}{(1+i)^n}$$

[3] A number of refinements, such as how to deal with intangibles, are ignored here in the interest of brevity. See Maass and others (*5*) and McKean (*6*) for more extended discussions.

[4] These topics are usually given extended treatment in textbooks of engineering economy. See, for example, Grant and Ireson (*2*). An excellent introduction to the issues is in Hirshleifer, DeHaven, and Milliman (*3*, ch. 6 and 7).

where V_0 = present value of the investment,

s_t = net benefits (benefits less capital and operating costs) attributable in year t to the investment,

i = rate of discount, and

n = last year in which the investment has any effect.

Other formulas exist which are related to or approximations of the one given, each of which will also involve the same elements, benefits, costs, and a discount rate. (The most familiar alternatives are the benefit-cost ratio and the internal rate of return formula.) The practical application of any such formula requires (a) predicting physical inputs and products associated with each time period over the life of the project, (b) assigning dollar values to these, and (c) selecting a discount rate.

Predicting outcomes. — The task of predicting inputs and products for a proposal is beyond the economist's province. However, it is important to note that those inputs and products relevant to evaluating a particular proposal are only those *incremental* to the proposal. Analysis may be facilitated with a tabular form that explicitly lists alternative investment possibilities together with their incremental contributions to benefits and costs.[5]

Costs and benefits. — The costs are the actual amounts spent on additional physical equipment, labor, and materials. Charges for interest, depreciation, or debt amortization should be omitted, for interest and depreciation are considered in the formula, while amortization of debt is not directly relevant to the economic desirability of a project. Many Federal cost-sharing programs are available for seepage-control projects. In these projects, the individual or agency may charge only its incremental cost, net of the subsidy.

Seepage-reduction investments generate many benefits. Of primary interest, of course, is the water "produced" or saved for productive use. Other benefits can arise from reduction in irrigation labor requirements, weed control and other canal maintenance expenditures, or controlling of waterlogging on adjacent lands.

Valuing the water savings should be performed on the principle of "alternative (or opportunity) cost" (best alternative use or source). The opportunity cost in this case is the *added* cost of the least expensive alternative source of water or the value in the marginal alternative use, whichever is the lesser. For example, a farmer evaluating seepage-reduction measures should value the water saved at the cost of additional replacement water. This might be equivalent to the variable costs per unit of pumping groundwater to secure the additional water, or the cost per unit of such water delivered to his headgate from an irrigation district.

An exception to this rule may occur. In the event that no additional water is available, say, due to limited water rights, the value in use must be estimated. Estimating the value productivity of water is a technically difficult task. Two techniques are available, estimating the production function by statistical analysis or by budgeting out a residual value. (The technique of residual estimation is time consuming and often of doubtful accuracy, because it requires all other resources be correctly priced in the analysis, while data for application of the statistical procedure may be difficult to obtain.)

Similar principles apply to the valuation of other benefits. That is, such savings as from reductions in weed or waterlogging problems should be valued on the basis of the least-cost means of achieving the reduction, or the net value of such control, whichever is the lesser.

Discount rate. — Since income received now is worth more than a similar amount of income at some future time, proper investment evaluation must discount the streams of net income to their present value. The present value of a stream of income is extremely sensitive to the rate of discount (or interest). Hence, it must be chosen correctly.

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[5] Such a format is used to display the analysis of alternative canal-lining techniques by Wineland and Lucas (*8*), although I do not altogether endorse their analysis and conclusions.

The rate of interest is viewed in economic theory as being a balance of considerations of time preference and productivity of capital. The market rate of interest is a measure of the aggregate appraisal by all demanders and suppliers of capital in the market of the rate of time productivity of capital and the rate of preference between present and future resources. If too low a rate is chosen for project evaluation, capital will be allocated to projects where the market rate indicates that producers and consumers value the capital more highly for purposes of private consumption or other alternative investments. The contrary case in which discount rates are set too high results in the opposite inefficiency.

Economists have been sharply critical of the prevailing practice of using the market rate on long-term Government bonds at the discount rate. One group of critics holds that the Treasury borrowing rate does not represent the true cost to the Government, since it ignores corporate and individual income taxes foregone as a result of borrowing by the Government to finance programs (7).

Another view holds that investments should be discounted directly on the basis of what is foregone; namely, the return that could have been earned on investments of equivalent risk in the private sector of the economy when the decision is made to commit resources to the public sector. In either case, rates of 7½ percent or more are advocated, which are substantially above that commonly applied in the evaluation of public water resources development projects in the past. (The President's 1968 budget message indicated that a higher rate would be applied henceforth, although I have not yet seen the final recommendation of the Water Resources Council.)

Sensitivity analysis. — Predictions of the physical benefit and cost items for a long period in the future are necessarily uncertain. It is, therefore, useful to the final decision maker to have sensitivity analyses performed on the key variables. Sensitivity analysis refers to determining the effect on the value of an investment of a change or error in one of the variables used in the evaluation. Some variables may change by substantial amounts without influencing the conclusions. Others, however, may exist to which the present worth is highly sensitive. Each variable should be tested to see how sensitive the decision is to variations from the best estimate for the value of the variable and the results used in the final decision-making process.

To summarize this section in less technical terms, the desirability of investing in a particular seepage-reduction measure depends upon (a) the value of the water saved and the associated cost reductions and productivity gains, (b) the cost of the materials and other resources used, (c) the effectiveness of the control measure over time all as compared with possible alternative measures, as well as (d) the rate at which future net benefits are valued at the present time.

CONCLUSION

Seepage-reduction investments appear to present no special conceptual problem. Therefore, I concentrated on reviewing general investment principles, emphasizing that decisions which effectively represent the client's interest (whether the client is a private individual or the general public) should be based on the selection of appropriate choice criteria and models, should be based on consideration of only those cost and benefit items incremental to the decision, and should be based on appropriate valuation procedures and discount rates.

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DISCUSSION

SESSION IV — SEEPAGE DETECTION AND EVALUATION

D. G. Shockley: Can salt penetration method be used to measure seepage into sloping canal banks?

H. Bouwer: We have not tried this in the field, but I see no reason why this should not be possible. The only difficulty may be to keep the salt in place for a sufficient period of time, particularly if the banks are steep. However, with the burlap-sack technique it should be possible to maintain sufficient contact time. Since the wetted perimeter of the channel is an equipotential line, the electrical conductivity probe should be inserted at a right angle to the slope so that it coincides with the initial direction of the streamlines.

M. F. Katzer: Is there danger of causing short circuiting along the stem of the probe when it is inserted into the canal bottom?

H. Bouwer: The probe is not in the bottom material while the salt is moving down to the depth where it is measured. Therefore, there is no chance for short circuiting. The probe is inserted only when the peak of the salt wave is expected to have reached a depth that can be conveniently and accurately measured, for example, a range of 2 to 6 inches. A few trial insertions are made at different times after the salt application to determine when the peak will have reached this depth range. Then, the probe is inserted in succession at a number of points to determine how far the salt wave has actually penetrated. The rate of insertion of the probe is about 10 inches per minute. Thus, measurements are completed in about 10 to 30 seconds, depending on the depth of the salt wave.

A. Halderman: Could variations in water temperature have affected seepage rate comparisons?

C. E. Brockway: Water temperatures in this canal probably did not affect the seepage comparisons between ponding rates and seepage meter rates since the comparisons were made at nearly the same time. Temperature variations could account for some differences between spring and fall seepage rates.

W. A. Lidster: How did the tests taken in November compare with seepage loss during the irrigation season?

C. E. Brockway: Seepage loss was not measured during the irrigation season because the canal could not be taken out of service and we were not able to obtain seepage meter measurements in the operating canal.

H. Glenn: Infrared shows presence of water or moisture, and without time factor, does not directly correlate with seepage rate. Interpretation is extremely important.

T. Ory: It is true that while infrared imagery may record an indication of abnormally high-moisture content, it is not a positive seepage-detection technique, and has no direct correlation with seepage rate. However, it appears possible that the use of a calibrated infrared scanning system, in combination with conventional photography and flown sequentially to establish a time base, may prove a reliable method of detecting seepage. Moreover, measurements of the magnitude of the thermal anomaly associated with seepage, in terms of temperature differential and area of extent, may provide information on seepage rate. Since the technique has not yet been tested on a real seepage problem, definitive data on its reliability and limitations simply are not available.

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